SEWAGE AND INDUSTRIAL WASTES

(Formerly Sewage Works Journal)

JOURNAL of the FEDERATION OF SEWAGE AND INDUSTRIAL WASTES ASSOCIATIONS

Special Features

Flow Path in Sedimentation—Fitch

Westchester County Problems—Griffin and Brown

Sewage Effluent Use—Skulte

Treatment of Petroleum Wastes—Baker and Wenger

Garbage Grinding at Jasper—Wraight

Cabin Cruiser Wastes—Ingram

JANUARY, 1956—VOL. 28, No. 1

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Please remit directly to your Member Association Secretary immediately upon receipt of the first dues notice from him.

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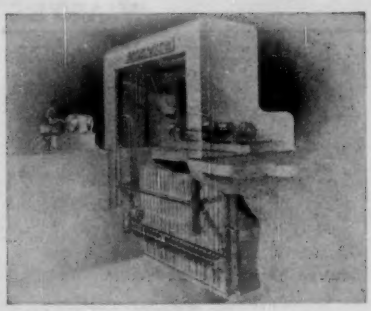

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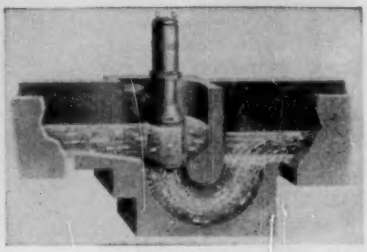

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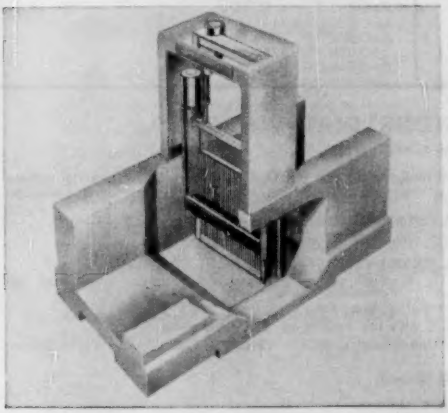

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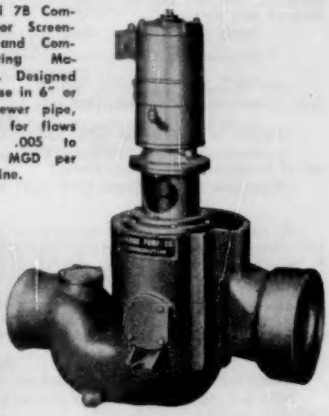

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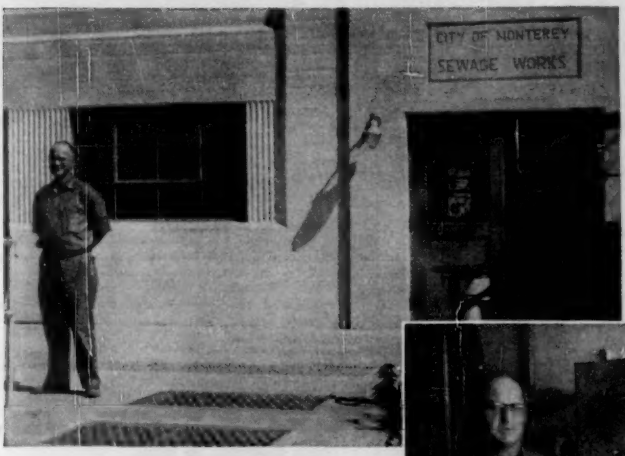

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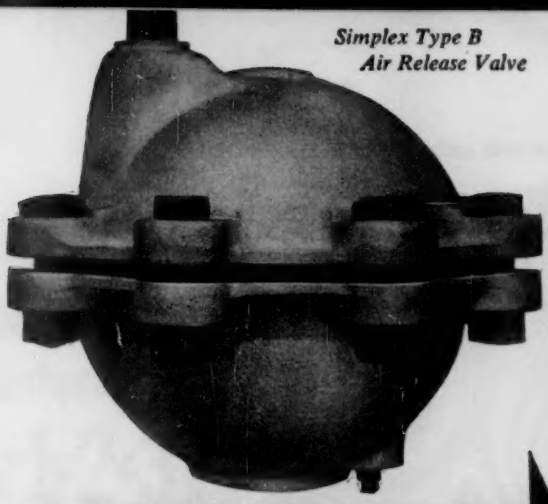

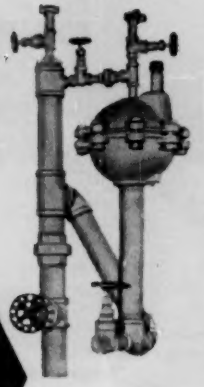

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Texas Water and Sewage Assn.	Texas A & M College College Station, Tex.	Mar. 5–9, 1956
New Jersey Sewage and Industrial Wastes Assn.	Hotel Traymore Atlantic City, N. J.	Mar. 14–16, 1956
Arkansas Water and Sewage Conference (*Silver Anniversary*)	Marion Hotel Little Rock, Ark.	Mar. 19–21, 1956
Kansas Sewage and Industrial Wastes Assn.	Jayhawk Hotel Topeka, Kansas	Apr. 4–6, 1956
Montana Sewage and Industrial Wastes Assn.	Murray Hotel Livingston, Montana	Apr. 5, 1956
Arizona Sewage and Industrial Wastes Assn.	Buena Vista Hotel Safford, Arizona	Apr. 5–7, 1956
Virginia Industrial Wastes and Sewage Works Assn.	Natural Bridge Hotel Natural Bridge, Va.	Apr. 19–20, 1956
California Sewage and Industrial Wastes Assn.	Santa Rosa Hotel Santa Rosa, Calif.	Apr. 25–28, 1956
Maryland-Delaware Water and Sewage Assn.	Willard Hotel Washington, D. C.	May 10–11, 1956
Michigan Sewage and Industrial Wastes Assn.	Whitcomb Hotel St. Joseph, Mich.	May 14–16, 1956
Ohio Sewage and Industrial Wastes Treatment Conference	Hotel Carter Cleveland, Ohio	June 13–15, 1956
Central States Sewage and Industrial Wastes Assn.	Northland Hotel Green Bay, Wis.	June 20–22, 1956

TWENTY-NINTH ANNUAL MEETING

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in conjunction with

California Sewage and Industrial Wastes Association

Statler Hotel, Los Angeles, California

October 8–11, 1956

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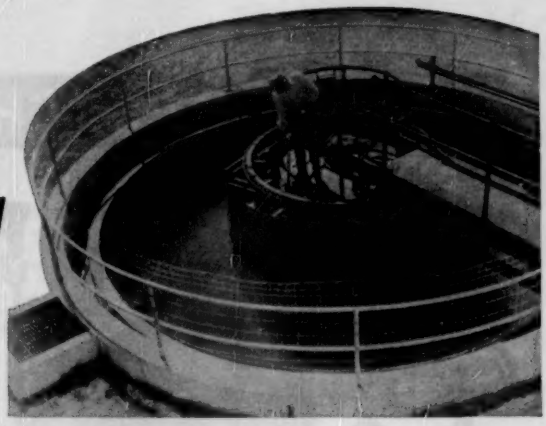

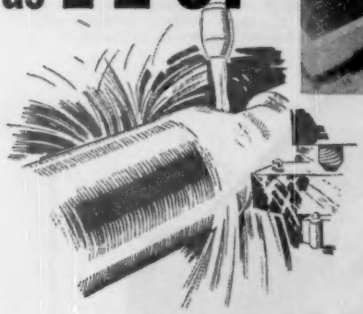

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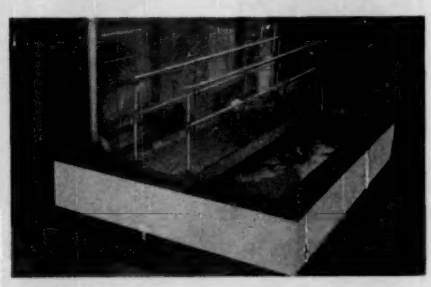

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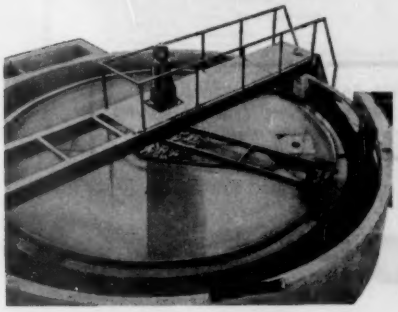
3. PRIMARY CLARIFIER: For recovery of oil and removal of settleable solids when two stage treatment is required.



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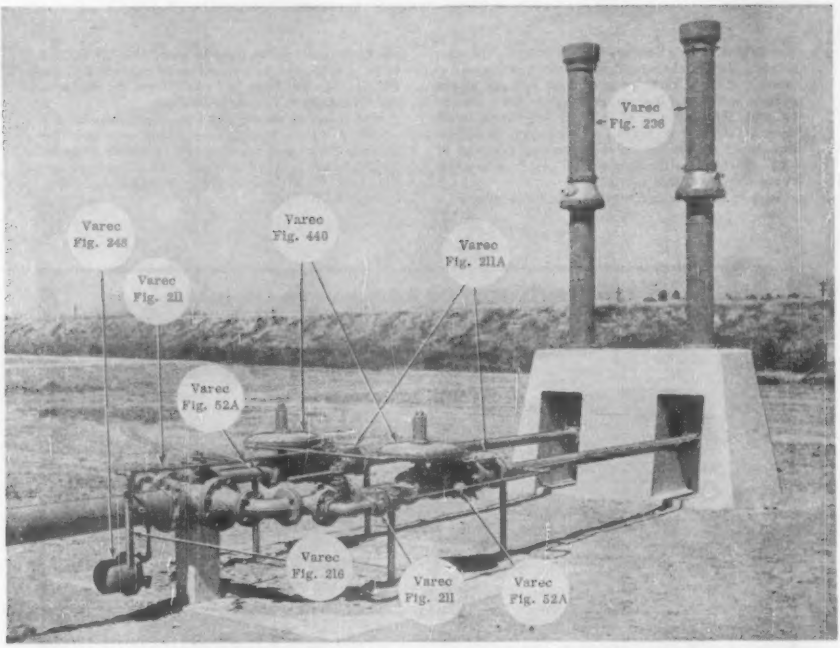
Bureau of Water, Department of Public Works
PLYMOUTH, Massachusetts
Plymouth Gas Light Company
POTTSVILLE, Pennsylvania
Pottsville Water Company
PROVIDENCE, Rhode Island
Providence Gas Company
QUEBEC, Canada
Quebec Power Company, Gas Division
READING, Pennsylvania
Bureau of Water
RICHMOND, Virginia
Department of Public Utilities (Gas)
RICHMOND, Virginia
Department of Public Utilities (Water)
ROCHESTER, New York
Rochester Gas & Electric Corp.
SACRAMENTO, California
Division of Water & Sewers
ST. JOHN, New Brunswick
Water & Sewerage Department
ST. LOUIS, Missouri
Department of Public Utilities, Water Division
SALEM, Massachusetts
North Shore Gas Company
*SPRINGFIELD, Illinois
Central Illinois Light Company
SYRACUSE, New York
Water Division, Department of Engineering
TORONTO, Ontario
The Consumer's Gas Co. of Toronto
TROY, New York
Department of Public Works
UTICA, New York
City of Utica, Board of Water Supply
WHEELING, West Virginia
City of Wheeling Water Department
WILMINGTON, Delaware
Wilmington Water Department
WINCHESTER, Virginia
Water Department
WINSTON-SALEM, North Carolina
Water Department
YORK, Pennsylvania
York Water Company
ZANESVILLE, Ohio
Water Department

*New Members in 1955

SERVES FOR CENTURIES

Safe Handling of Surplus Sewage Gases



VAREC

SEWAGE GAS CONTROL and SAFETY DEVICES

Modern Sewage Treatment Plants, like other processing plants, require equipment specifically designed to maintain conditions which will produce maximum operating efficiency.

VAREC equipment has become recognized as "standard" for safe and efficient sewage gas handling. It is simple, foolproof and dependable — yet designed to meet the highest engineering standards of accuracy, sensitivity and reliability.

The photograph above illustrates a typical VAREC control and waste gas burner installation for safe disposal of excess gases.

For additional details on this installation, or information on other available VAREC equipment, contact your nearest VAREC representative or address:

"Varec"

S-11

VAPOR RECOVERY SYSTEMS COMPANY
2820 N. Alameda St., Compton, Calif.
Cable address: VAREC COMPTON Calif. (U.S.A.) All Codes

Available from Authorized Sewage Equipment Agents throughout U.S. and Canada.

THE SUN'S HEAT AND GIANT REFLECTORS may convert cosmic energy into power that will propel space ships to distant planets.

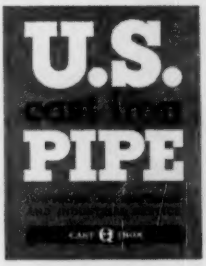
100 years from now...

WE MAY "SIGHT SEE" ON MARS

A century from now, when undreamed-of marvels transform our way of life, civilization will still rely on one familiar public servant: the cast iron pipe laid today will continue to carry water and gas to the homes of the future.

Proof? 69 American cities and towns are still using cast iron water and gas mains over a century old. And today's modernized cast iron pipe . . . centrifugally cast and quality controlled . . . is even stronger, tougher, more serviceable.

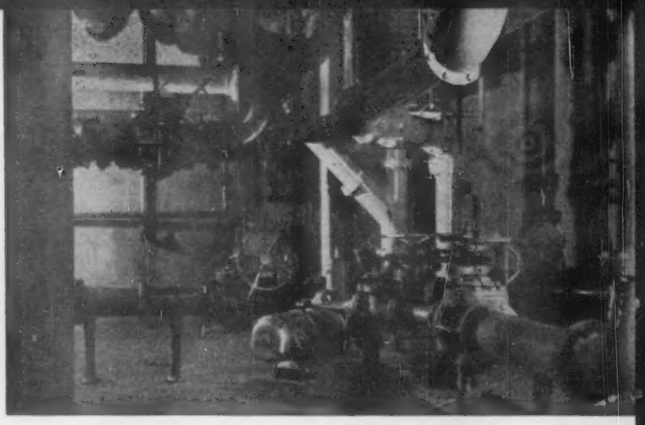
U.S. Pipe is proud to be one of the leaders in a forward-looking industry whose service to the world is measured in centuries.



U. S. PIPE AND FOUNDRY COMPANY, General Office: Birmingham, Alabama

WHOLLY INTEGRATED PRODUCER FROM MINES AND BLAST FURNACES TO FINISHED PIPE.

versatile, dependable ..at lowest cost



ROCKWELL-Nordstrom VALVES

for any wastes service

The photo above shows Rockwell-Nordstrom valves on sump discharge and supernatant lines at one of the country's largest sewage disposal plants. 40 years of experience have proven that Rockwell-Nordstrom valves allow complete versatility of installation, dependable flow control on any wastes service, and lowest maintenance cost.

Rockwell-Nordstrom valves are ideal for wrench, chain wheel, gear, floor stand and power operation because *quarter-turn operation* is two to five times faster than ordinary valves. Streamlined flow passages minimize pressure loss on low pressure sewage gas as well as heavy sludge. The *tapered plug* is "cushioned" on its seat by pressurized lubricant for instant, smooth operation. Lubricant in Rockwell-Nordstrom valves also eliminates metal-to-metal wearing friction. Maintenance and down time costs are lower than any other valve.

Write for more information on how you can have dependable, trouble-free valve service at lowest cost with Rockwell-Nordstrom valves.

40th ROCKWELL YEAR

of lubricated plug valve leadership

Canadian Valve Licensee: *Peacock Brothers Limited*

ROCKWELL-
Nordstrom
VALVES

Lubricant Sealed for Positive Shut-off

Rockwell Manufacturing Company
Pittsburgh 8, Pennsylvania

☐ Please send me Bulletin C-5200.
☐ Please have your sales engineer call.

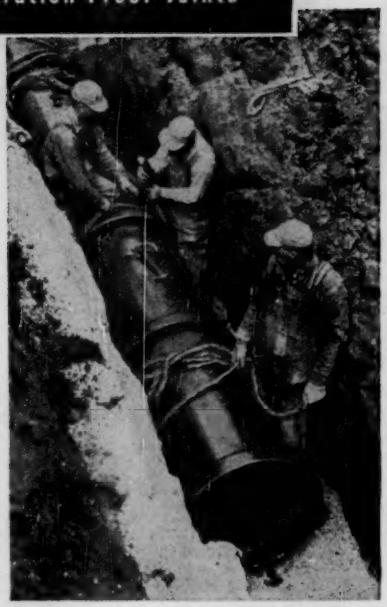
Name_____ Title_____

Address_____

City_____ Zone____ State_____

For Trouble-Free Service, Infiltration-Proof Joints

COLUMBIA, MO. Specifies CLAY PIPE



COLUMBIA, MISSOURI'S latest municipal sewerage improvement project called for more than 42,000 feet of 24-inch to 36-inch Vitrified Clay Pipe. A large section of the project is located in the city's attractive university district, where officials are especially interested in avoiding unsightly repair jobs in the years ahead. Other parts of the line were laid adjacent to a creek, making leak-proof, infiltration proof joints a "must." With these requirements in mind, officials in charge of the project specified Vitrified Clay Pipe . . . the *only* pipe that never wears out. A wide variety of new jointing methods, developed through industry-wide research, insure *absolutely tight* lines—even in high-water-table areas. For *permanent* service, always specify and install Clay Pipe. *It's guaranteed for half a century.*

FOR COLUMBIA, MO.—S. E. Grundler, City Engineer

CONSULTING ENGINEERS—Black and Veatch, Kansas City, Mo.

CONTRACTORS: B & G Construction Company, Carthage, Mo., Supt. Roy E. Fuller, and Foreman Lewis Dye.

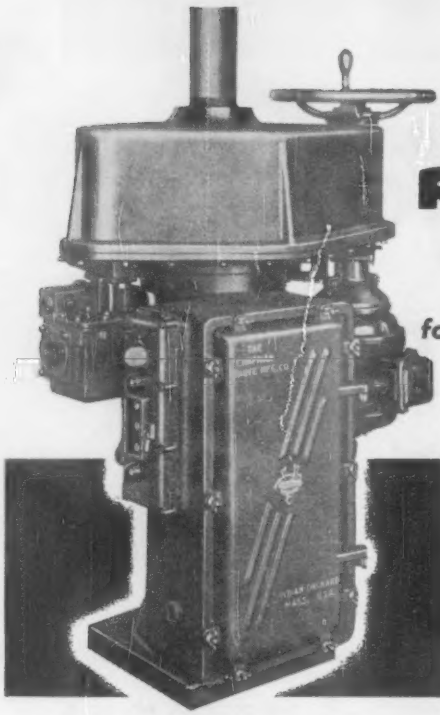
Vitrified CLAY PIPE

NATIONAL CLAY PIPE MANUFACTURERS, INC.
1820 N. Street, N.W., Washington 6, D. C.

206 Connally Bldg., Atlanta 3, Ga.
100 N. LaSalle St., Rm. 2100, Chicago 2, Ill.
703 Ninth & Hill Bldg., Los Angeles 15, Calif.
311 High Long Bldg., 5 E. Long St., Columbus 15, O.

C-156-IA

Progress in Public Health - Through Clay Pipe Research



Simple and RUGGED...

CHAPMAN'S MOTOR UNIT

for Valves, Floorstands and Sluice Gates

Floorstand equipped with Motor Unit . . . control panel, motor, limit switch and push button station.

Simple, durable mechanism of Chapman's Motor Unit. Handwheel remains stationary during motor operation.

Chapman Motor Units have fewer components than any other units, providing greater dependability, much lower maintenance costs.

There are only two pinions and two gears in the motor reduction train: drift is eliminated and lash is negligible. Your valves seat tight, but not too tight. Operation is *always* smooth and trouble-free.

You can mount the Chapman Motor Unit in any position, at any angle required. The rugged stub-tooth gears require no grease or oil bath . . . yet run smoothly, quietly, with minimum wear.

Weatherproof and steam tight, Chapman's floorstand units are shipped completely wired, ready to connect to your power leads, for easy, rapid installation. This modern motor unit is dependable and economical. Send for *new* catalog No. 51.

The CHAPMAN Valve
Manufacturing Company
INDIAN ORCHARD, MASSACHUSETTS

Sure way to clarify industrial wastes

from this to this

Glass graduate at left shows waste as it flows into settling tank at chemical plant. Mound of sludge filter cake in center represents solids taken from liquid (right) before discharge into sewerage system.

LINK-BELT Circuline Collector concentrates sludge fast in treating corrosive waste at Ohio chemical plant

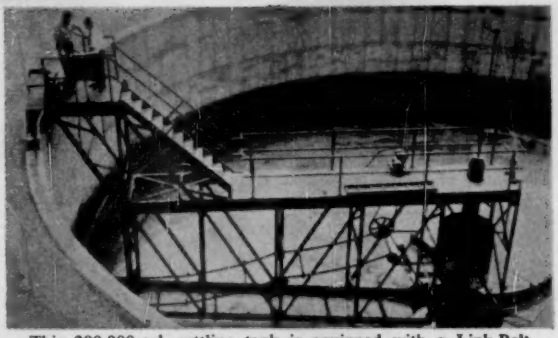
In Cincinnati, surcharges are levied on industrial wastes that exceed maximum legal strength when discharged to city sewers. At the St. Bernard, Ohio plant of Toms River-Cincinnati Chemical Corporation, Cincinnati Division, corrosive and abrasive acid wastes make treatment prior to discharge to the city sewerage system the most economical disposal method.

As in municipal water and sewage plants and industries throughout the country, Link-Belt had a hand in the solution. A Circuline Collector in the settling tank gathers settled sludge so gently that minimum agitation and maximum concentration occurs. This results in high tank efficiency in removing solids and minimum costs to dewater the sludge. It is meaningful that in seeking maximum efficiency and maximum sludge density, Link-Belt equipment was recommended by the consultants, R. L. Woolpert Co. of Dayton.

In the same way, our engineers will help work out answers to your specific needs, collaborating with your engineers, consultants and chemists. Call the Link-Belt office nearest you.

LINK-BELT

SANITARY ENGINEERING EQUIPMENT



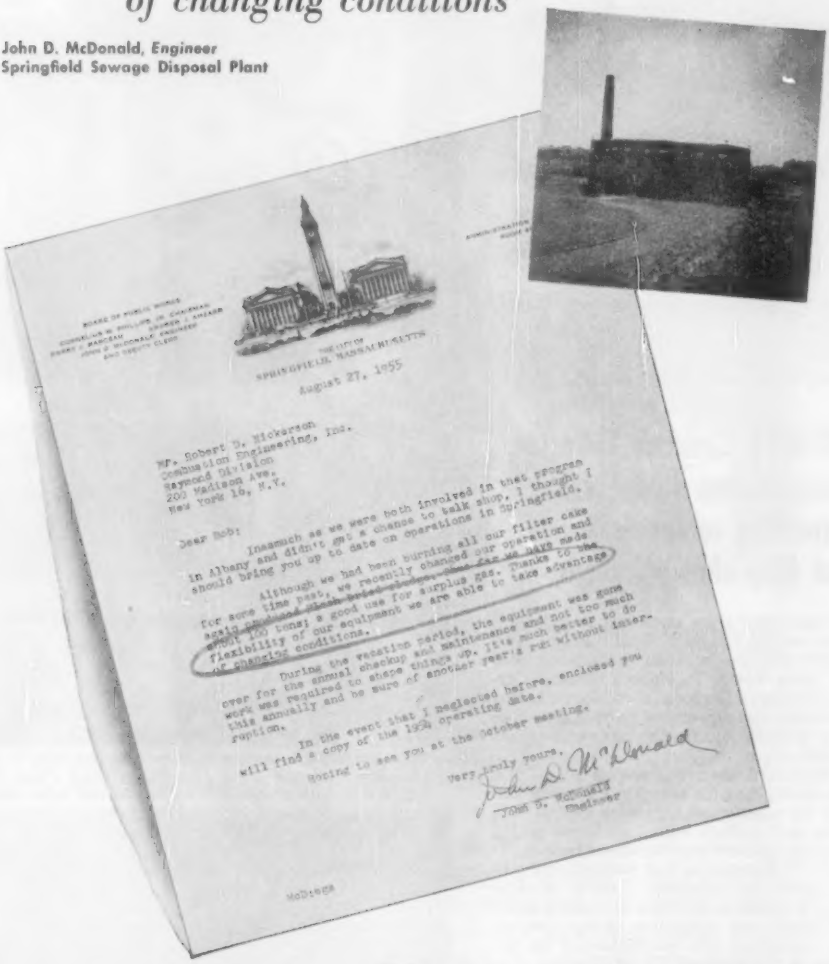
This 200,000-gal. settling tank is equipped with a Link-Belt Circuline Collector using Link-Belt Promal chain. Power is supplied through a Link-Belt worm gear and roller chain drive.

After sludge is concentrated in two 13,500-gal. tanks, it is dewatered on a coil-spring vacuum filter and carried from filter by Link-Belt 18-in. wide belt conveyor to be trucked to a dump.

LINK-BELT COMPANY: Executive Offices, 307 N. Michigan Ave., Chicago 1. To Serve Industry There Are Link-Belt Plants and Sales Offices in All Principal Cities. Export Office, New York 7; Canada, Scarboro (Toronto 13); Australia, Marrickville, N.S.W.; South Africa, Springs. Representatives Throughout the World. 13,272

"Thanks to the flexibility of our equipment *we are able to take advantage of changing conditions"*

John D. McDonald, *Engineer*
Springfield Sewage Disposal Plant



August 27, 1959

Mr. Robert D. Hickerson
Combustion Engineering, Inc.
Raymond Division
200 Madison Ave.
New York 16, N.Y.

Dear Bob:

Inasmuch as we were both involved in that program in Albany and didn't get a chance to talk shop, I thought I should bring you up to date on operations in Springfield.

Although we had been burning all our filter cake for some time past, we recently changed our operation and again [illegible] tons; a good use for surplus gas. Thanks to the flexibility of our equipment we are able to take advantage of changing conditions.

During the vacation period, the equipment was gone over for the annual checkup and maintenance and not too much work was required to shape things up. It's much better to do this annually and be sure of another year's run without interruption.

In the event that I neglected before, enclosed you will find a copy of the 1958 operating data.

Hoping to see you at the October meeting.

Very truly yours,
John D. McDonald
John D. McDONALD
Engineer

McDingo

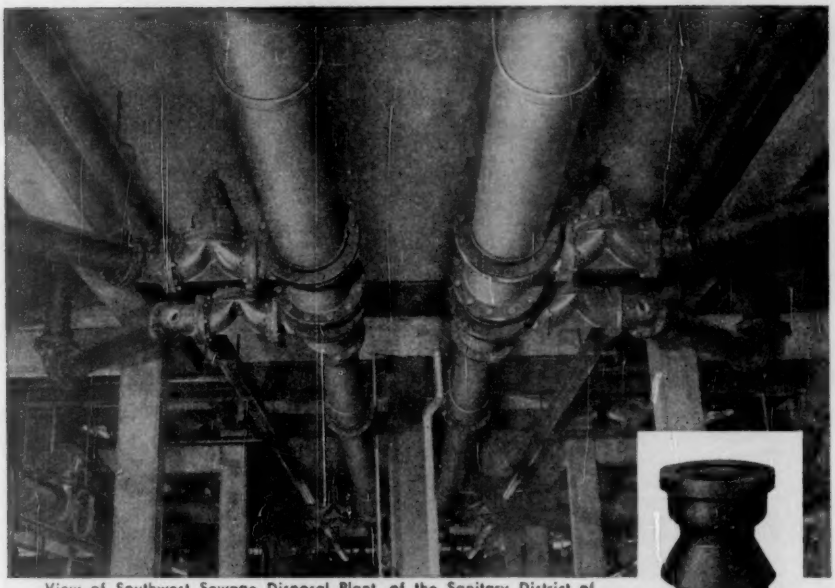
C-E

COMBUSTION ENGINEERING
RAYMOND DIVISION
1315 North Branch Street, Chicago 22, Illinois
Eastern Office: 200 Madison Avenue, New York 16, N. Y.
Western Office: 510 West Sixth Street, Los Angeles 14, Calif.
Canada: Combustion Engineering-Superheater Ltd.

The C-E Raymond Flash Drying System is the most flexible ever developed. It is available in designs that permit *either* flash drying sewage sludge to a marketable soil conditioner . . . *or* incinerating it to sterile ash — in any proportion and under complete control. In either case, high temperature deodorization of stack gases may be included if you wish. For details call your nearest C-E Raymond office.

B-866A

ALSO FLASH DRYING AND INCINERATION SYSTEMS FOR INDUSTRIAL WASTE DISPOSAL



View of Southwest Sewage Disposal Plant, of the Sanitary District of Chicago, Stickney, Illinois . . where Clow Cast Iron Pipe was furnished to meet exact requirements.

Clow cast iron pipe, fittings, valves and specials for sewage plant replacements or new installations

• James B. Clow & Sons with their many years of experience as manufacturers of Cast Iron Pipe and Special Castings for Filtration and Sewage Disposal Plants know the difficult piping problems that continually face the sanitary engineer. Clow shops are completely equipped with the latest machinery. Clow's skilled workmen will produce pipe and specials to your exact specifications. Pipe can be furnished flange and flange, flange and bell, flange and plain end or with mechanical joint and plain end—in sizes 3 through 24 inches in laying lengths up to 18 feet.

CLOW (Threaded) Cast Iron Pipe

Clow also manufactures (threaded) Cast Iron Pipe in steel pipe sizes 3 through 10 inches which has the same outside diameter as common steel pipe. This pipe can be cut, threaded and fitted on the job with ordinary tools of the piping trade.

JAMES B. CLOW & SONS ☆

201-299 North Talman Avenue • Chicago 80, Illinois

Subsidiaries:
Eddy Valve Co., Waterford, N. Y.
Iowa Valve Co., Oskaloosa, Iowa

REMOVE GRIT

Easily—Thoroughly—at Low Cost

COMPLETE EQUIPMENT

to do the complete job!

PROVIDES CONSTANT OPTIMUM VELOCITY
POSITIVE ADJUSTMENT FOR VARYING FLOWS
HYDRO-DYNAMIC WASHING
AUTOMATIC DRAINING AND REMOVAL

FLOW CONTROLS—exclusive Camp Channel Section and Regulator* maintain constant optimum velocity for effective grit settling, regardless of rate of flow through plant. Velocity can be easily and instantly adjusted to meet changing conditions. Minimum head loss.

*Patented

CONVEYOR—single strand, continuous drag type. Flights carry the settled grit from channel bottom to hopper for washing and removal.

HYDROWASH UNIT—washes grit free of organic matter by producing a turbulent downflow recirculation and agitation of liquor in grit hopper. Organics returned to hopper.

ELEVATOR—totally enclosed, tube type, for slow and continuous removal of washed grit from the hopper. Grit drains as it is being elevated through the tube.

Write for Bulletin No. 249B

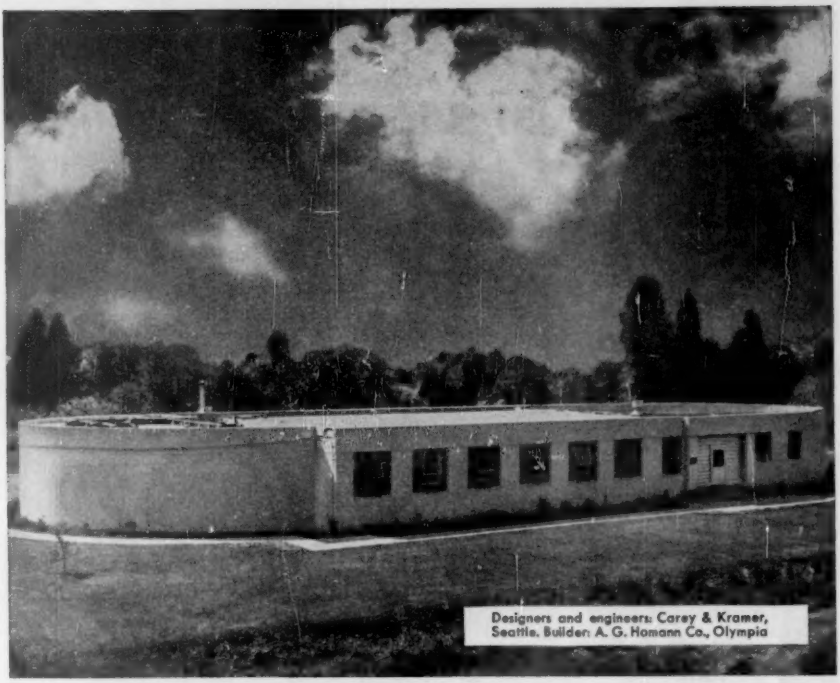
Technical Supplement "CR", containing complete design data on the AMERICAN CAMP GRIT CHANNEL *and* REGULATOR, *is available on request with Bulletin 249B.*

AMERICAN WELL WORKS

IN OUR 87TH YEAR
110 North Broadway
AURORA, ILLINOIS

Pumping, Sewage Treatment, and Water Purification Equipment
RESEARCH • ENGINEERING • MANUFACTURING

Offices: Chicago • New York • Cleveland • Cincinnati • Kansas City • Sales Representatives throughout the World



Build Functional and Distinctive Sewage Treatment Plants with Architectural Concrete

This Centralia, Wash. sewage treatment plant exemplifies how architectural concrete combines modern functional qualities with distinctive appearance.

Centralia has a combined sanitary and storm sewer system. In the rainy season, with sewage getting only primary treatment, the plant's capacity is 4½ m.g.d. As a complete treatment plant the rest of the year its capacity is 2 m.g.d.

Architectural concrete is an ideal construction material for sewage treatment plants. Its clean, neat appearance is a symbol of cleanliness and good municipal housekeeping. It makes the structure a long-lasting asset and a source of community pride. In addition, it offers:

1. UNUSUAL DURABILITY. It resists such destructive agents as severe weathering, fire, decay and sewage gases.

2. GREAT STRENGTH. It can be designed economically to withstand all vertical and horizontal forces.

3. REAL ECONOMY. Moderate first cost, low maintenance cost and long life result in true ***low-annual-cost*** service.

Investigate architectural concrete's advantages for sewage treatment plants. Ask for free literature, distributed only in U.S. and Canada. Address Dept. A6-76.

PORTLAND CEMENT ASSOCIATION

33 W. Grand Avenue
Chicago 10, Illinois

A national organization to improve and extend the uses of portland cement and concrete . . . through scientific research and engineering field work

THE COILFILTER

Sanitary Engineers' Notebook

MEMO:

The sludge to be dewatered on the COILFILTER is usually obtained from a sump or storage tank. It must be lifted to the sludge conditioning tank by equipment which is reliable and trouble-free. This equipment must deliver sludge at a rate proportional to its speed, regardless of the level in the sump, and should have a variable speed drive and a revolution counter, so that it will be an accurate metering device.

The Komline-Sanderson dual-valve sludge pump (patents pending) is the most reliable and least expensive equipment fulfilling these requirements, and is available in either diaphragm or plunger types. This pump has two ball check valves in series on both suction and discharge sides. If one valve is unseated due to foreign matter in the ball chamber, the other ball valve will still keep the pump operating properly. The foreign matter is usually flushed through the pump, without interruption of operation.

A number of these dual-valve sludge pumps have been in operation for several years and the freedom from clogging is almost beyond belief.

Their development and application represent a fundamental step forward in easy-to-operate sludge dewatering installations.

Sludge conditioning will be discussed in the next issue of Sanitary Engineers' Notebook.

KOMLINE-SANDERSON ENGINEERING CORPORATION
PEAPACK, NEW JERSEY

Manufacturers of COILFILTER *sludge vacuum filters*

THE LINE THAT HELD

At Waterbury, Conn., this Lock Joint Reinforced Concrete Cylinder Pipeline, undermined by flood waters, gave trouble-free service, alleviating a critical water shortage.

Although sections of this 48" Lock Joint water supply line for Hartford, Conn. were uncovered by the flood, it continued to give satisfactory service.

The destiny of England once hung on Wellington's "thin, red line" at the Battle of Waterloo. Daily your community's safety from fire and epidemic depends, to a large degree, on its water supply and sewage lines.

When the backwash of Hurricane Diane inundated western New England, the many Lock Joint installations in the area stood firm. These lines, in some cases the main source of water supply or of sanitary sewage disposal, continued to give trouble-free service even when uncovered and undercut by raging flood waters.

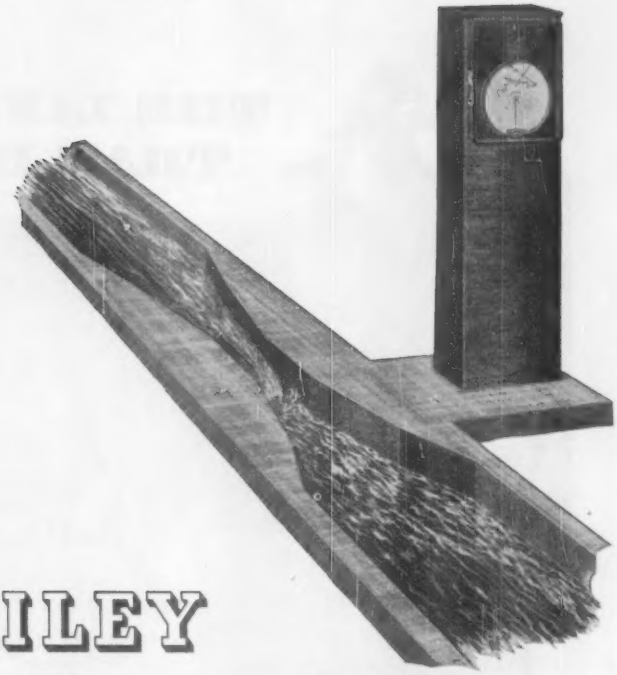
Such rugged pipe deserves your consideration. Its strength is evident, its initial high carrying capacity is permanent and its useful life is exceptionally long.

LOCK JOINT PIPE CO.

East Orange, New Jersey

Sales Offices: Chicago, Ill. • Columbia, S. C. • Denver, Col. • Detroit, Mich. • Hartford, Conn. • Kansas City, Mo.

Pressure • Water • Sewer • REINFORCED CONCRETE PIPE • Culvert • Subaqueous



BAILEY

Open Channel Meters

for trouble-free low cost measurement of

Sewage, Industrial Wastes, Sludge, Irrigation, Water Supply

Bailey Meters offer these advantages—

1. Easy to Install and Maintain
2. Retain Accuracy
3. Self Cleaning
4. Adjustable Capacities
5. Totalize Multiple Flows
6. Chemical Feed Control
7. Flow and Ratio Controls
8. Low Cost

For further information on how Bailey Meters can meet your needs, ask for Bulletin M22-5.

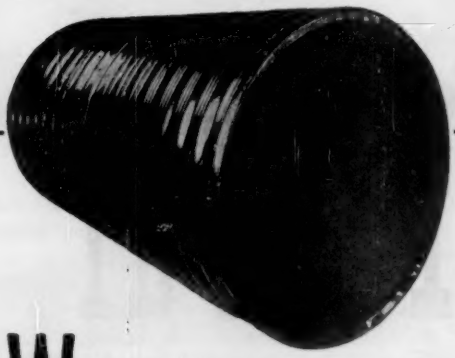
MU25

Bailey Meter Company

1066 Ivanhoe Road • Cleveland 10, Ohio

Meters and Controls for Sewage and Water

Venturi Tubes • Flumes • Weirs • Nozzles • Orifices • Direct Mechanical and Remotely Located Registers • Air-Operated, Electronic and Electric Controls



Armco
SMOOTH-FLO
Sewer Pipe

NOW an Armco Sewer for Every Need

All Armco Sewer Structures have the proved strength of corrugated metal design. Then it is simply a matter of selecting the type that will meet your exact needs most efficiently and economically.

For peak flow requirements— Armco SMOOTH-FLO Sewer Pipe

Armco SMOOTH-FLO Sewer Pipe combines top flow capacity, corrugated metal strength and watertightness. It has a spun bituminous lining that fills the corrugations. Available in bituminous-coated or ASBESTOS-BONDED to meet specific conditions.

A Type for Every Condition

Plain Galvanized Corrugated Metal Pipe—for normal sewer conditions.

Armco PAVED-INVERT Pipe—for severe erosive conditions.

Bituminous-Coated Galvanized Pipe—extra protection for moderately severe corrosive conditions.

Armco ASBESTOS-BONDED Pipe—for the most severe corrosion problems.

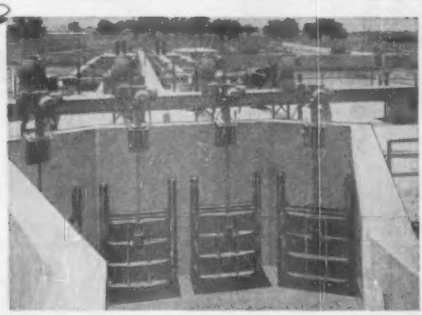
For sizes, gages and other data applied to your specific sewer problems, write us. Armco Drainage & Metal Products, Inc., 3136 Curtis Street, Middletown, Ohio. Subsidiary of Armco Steel Corporation.

Armco Sewer Structures

ARMCO

Plan with Quality...

Pekrul



4-36" Pekrul model 45A Sluice gates at Sacramento Sewage Treatment plant

City managers...engineers... contractors...know that the precision engineering and quality construction of Pekrul Gates means a *planned* water control project that will fulfill its purpose. It costs no more to plan with quality...specify Pekrul Gates on your next project.

PEKRUL GATE DIVISION

Write on your letterhead for NEW Pekrul Gate Catalog No. 55

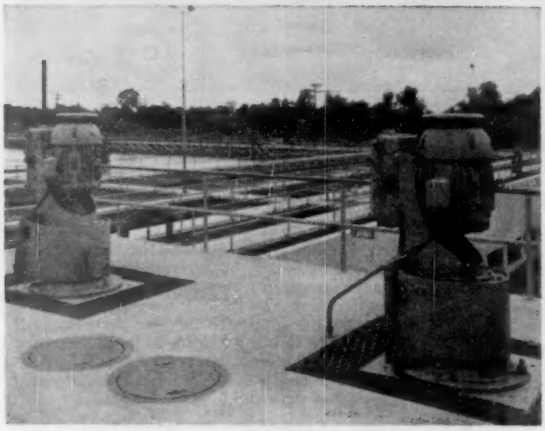
MORSE BROS. MACHINERY
DENVER, COLORADO

MANUFACTURERS OF PEKRUL GATES FOR

Flood Control · Sewage Disposal · Rearing Ponds · Recreation Pools
Levees · Reservoirs · Irrigation · Cooling Towers · Pumping Plants
Water Works · Oil Refineries · Steel Mills · Dams · Fish Hatcheries

for proper blending : JEFFREY RAPID MIXERS

are designed for efficient mixing of coagulants with raw water ahead of flocculation basins. Turbine-type impellers, alternate blades pitched, produce correct horizontal and vertical mixing for proper blend. The vertical shaft hangs from a motorized speed reducer placed directly over chamber. It is provided with a steady bearing at the bottom, flanged coupling, and anti-friction thrust bearing. For Catalog 833-A, write to Sanitation Sales Division, The Jeffrey Manufacturing Company, Columbus 16, Ohio.



Apron, Belt, Spiral Conveyors
Bar and Disc Type Screens
Biofiltration Plant Equipment
Bucket Elevators
Chains and Sprockets
Dry Chemical Feeders
FLOCTROLS
Garbage Grinders
Grit Collectors and Washers
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Scum Removers
Sludge Collectors
Sludge Elevators
Traveling Water Screens

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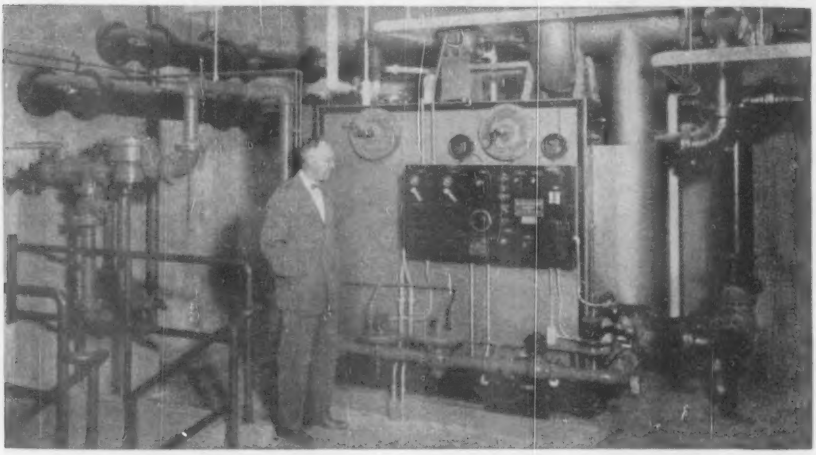
JEFFREY

CONVEYING • PROCESSING • MINING EQUIPMENT
TRANSMISSION MACHINERY • CONTRACT MANUFACTURING

G. W. Martin, Green Bay's Supt. of Sewage Treatment, checks new PFT Heater & Heat Exchanger



Green Bay selects PFT Digester Heater for operating economy, close control

In expanding its digesting operation, Green Bay, Wisconsin, once again chose PFT equipment for modern *controlled* digestion. In 1936, PFT had supplied the primary treatment plant with two Floating Covers for its 40′ digesters.

Key feature of the newly expanded plant is the PFT Heater & Heat Exchanger (750,000 B.t.u. per hr.). Located in the adjacent Control Chamber with all parts accessible, this compact unit heats sludge *externally* with no piping or equipment required inside the digesters.

Temperature of the sludge is maintained at optimum range by recirculating raw sludge or digestion tank contents (or a mixture of both) through the unit.

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SEWAGE AND INDUSTRIAL WASTES

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Contents

Sewage Works

Flow Path Effect on Sedimentation. By E. B. Fitch 1

Synthetic Detergents and the B.O.D. Test. By Waldron D. Sheets and George W. Malaney 10

Westchester County Solves Some Sewerage Problems. By Guy Griffin and James Brown 18

Discussion by James M. Brown 24

Research on Activated Sludge. VII. Efficiency of the Diffused Air System. By A. Pasveer 28

Irrigation with Sewage Effluents. By Bernard P. Skulte 36

Garbage Grinder Experiences, Jasper, Indiana. By Frank D. Wraight 44

Industrial Wastes

Solving a Large-Scale Waste Problem at Appliance Park. By Kenneth S. Watson and C. M. Fair 49

Biological Treatment of Petroleum Wastes. By Robert A. Baker and Roy F. Weston 58

Processing of Phosphorus Furnace Wastes. By John P. Horton, J. Douglas Malley and Harold C. Bays 70

Stream Pollution

Slime Formation in Polluted Waters. II. Factors Affecting Slime Growth. By H. Heukelekian and E. S. Crosby 78

An Investigation of the Treatment of Cabin Cruiser Wastes. By William T. Ingram 93

The Operator's Corner

Air Flotation Studies of Sanitary Sewage. By Thomas T. Hays 100

Discussion by A. W. Banister 106

Maintenance of the New York City Intercepting Sewer System. By John J. Rooney 107

Tips and Quips 112

Federation Affairs

Twenty-Eighth Annual Meeting Report 114

Proceedings of Member Associations

Pennsylvania Sewage and Industrial Wastes Association 30a

Missouri Water and Sewerage Conference 32a

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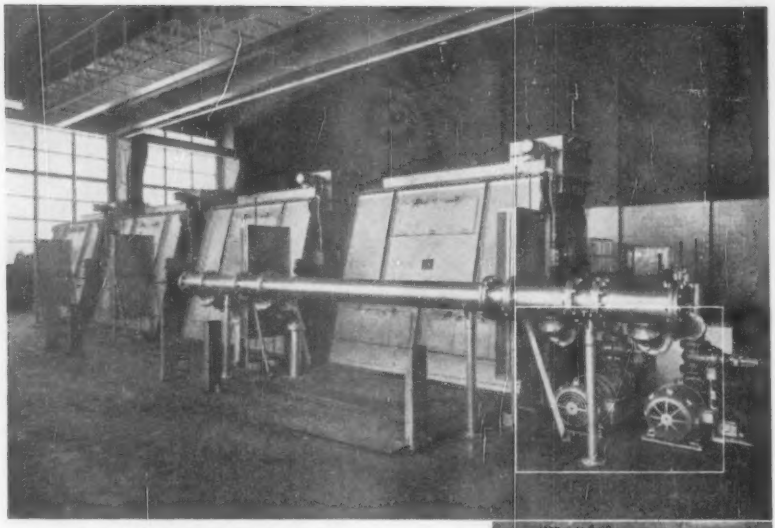
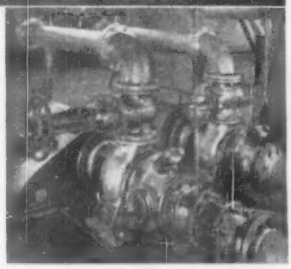


General view of screen room at Hyperion with Type S Screens at left and Disintegrators at right.

Screenings Handling Made Easy at Los Angeles Hyperion...



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Sewage Works

FLOW PATH EFFECT ON SEDIMENTATION

By E. B. Fitch

Director of Research, Dorr-Oliver Inc., Westport, Conn.

In 1904 Hazen (1) stated that the removal of silt from an "ideal" settling basin should be a function of the overflow rate and not the detention time. An ideal settling basin as defined by Camp (2) is one in which "the direction of flow is horizontal and the velocity is the same in all parts of the settling zone." No one has expected any practical basin to approach closely this ideal behavior. The intention was, as stated by Hazen, "First conditions much simpler than those which actually exist must be assumed, and from these simple assumptions the more complex conditions can be approached."

The concept of an ideal sedimentation basin is useful because, to the extent that the particles settle individually, each with a characteristic constant velocity, the sedimentation behavior of an ideal basin can be analyzed rigorously. The approach to the more complicated conditions actually existing is made by deducing amendments to the simple theory to account for various deviations from the ideal conditions. These deductions are necessary and instructive, but the hazard exists that the premises from which the deductions are made may not be completely valid. It would be safe to rely on a theory deduced only to the extent that each deduction could be validated by experiment or experience.

Some of the theoretically deduced conclusions published in the literature have proved controversial. A notable example is the conclusion that "long narrow rectangular sedimentation tanks are vastly superior in performance to radial flow sedimentation tanks." This conclusion is based, to a large extent, on the fact that long narrow tanks have been found experimentally to be more stable against short-circuiting as measured by dye tests, and on the postulate that such short-circuiting would be harmful to sedimentation.

In general, sanitary engineers have not yet accepted the long narrow rectangular sedimentation basin as an economically efficient design. At the same time the long narrow rectangular basin has been championed in the literature (3)(4). In view of this, and other points in which experience and the published theory do not completely conform, it seemed advisable to reexamine the theoretical development for limiting conditions or lack of generality.

An "ideal" basin must be one that can be analyzed rigorously. The more closely the ideal conditions approach reality, the fewer the amendments necessary to deduce practical behavior, and the fewer the chances to go astray. It is not difficult to analyze sedimentation mathematically under conditions less idealized than postulated by Hazen and Camp.

Analysis of "Ideal" Basin

First, it is assumed that each particle settles independently through and

with respect to the surrounding fluid, at a rate which remains constant. This assumption is known to be essentially valid for dilute suspensions in which flocculation or coagulation is not taking place during the settling period itself. It is also assumed that after a particle has settled out of a given flow, it will not be resuspended in it.

Second, a somewhat idealized tank or basin is postulated, in which mixing of flow paths is negligible. This is essentially equivalent to postulating streamline flow. The basin initially is assumed to be of any shape. The flow paths or streamlines may curve in any manner, and flow velocities are not assumed to be constant from point to point.

A conventional approach to solving for the removals in a basin would be to set up height or depth as the vertical axis or ordinate of a spacial coordinate system, with length as the second axis, and width as the third axis. Particles which had settled to the bottom of the sedimentation basin, while the flow was traversing the length of the basin, would be considered as removed. However, if curvature of the flow paths is to be considered, the flow may have an upward component of velocity at some points, and a downward component at other points. Also the flow may curve laterally. It thus becomes a difficult task to calculate the trajectory of a settling particle.

The entire difficulty is avoided in the following procedure by the expedient of transforming to a new coordinate system, relating to the flow rather than the spacial dimensions of the sedimentation basin. The trajectory of a settling particle is then reckoned relative to the flow net, rather than to the basin walls. The vertical axis of the new coordinate system measures the quantity of flow which passes above any given point, regardless of how the flow lines may incline or vary. Particles which settle to the bottom of

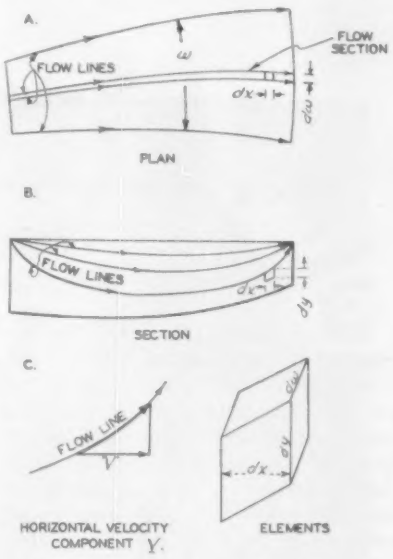

FIGURE 1.—Elements of a flow section showing plan and section and the velocity components.

the flow net are considered removed. This is the usual assumption since the bottom of the flow net and the bottom of the sedimentation basin must coincide.

A vertical section of the flow net of infinitesimal width is considered to extend from inlet to outlet, and is bounded by the flow lines. (Figure 1). The width dw may vary from the inlet to the outlet end of the flow section, since the flow lines may be expanding or contracting laterally, as in a circular sedimentation basin. The width is considered not to vary from the top to the bottom of the tank. This imposes a certain further restriction on the postulated flow pattern, but not a serious one. It requires only that the flow lines shall not be expanding or contracting differently at one vertical level than at another.

The flow through any infinitesimal cross section may be expressed as:

$$dq = V\, dw\, dy \qquad (1)$$

in which

q = the flow through the section;
V = the horizontal component of velocity, assumed to vary from point to point;
dw = the width of the flow stream at the cross section;
dy = the vertical height of the filament of the flow passing through the cross section.

The horizontal distance that a settling particle would travel while settling across a filament of flow is derived as follows:

$dy/dt = v$ = the settling velocity of the particle with respect to the fluid. (2)

Time is represented by t.

$$Vdt = dx \tag{3}$$

in which x = the horizontal component of distance along the flow line. Eliminating dt, between Eqs. 2 and 3 the equation becomes

$$dy = \frac{v}{V}dx \tag{4}$$

From Eqs. 1 and 4

$$dq = vdwdx \tag{5}$$

But

$$dwdx = dA$$

in which A = the projected or pool surface area across which the flow sweeps. Therefore,

$$dq = v\,dA \tag{6}$$

Integrating Eq. 6 over all filaments of flow traversed by a particle, assuming that the particle has traversed no flow at the time it has swept through no area, A.

$$\int_0^q dq = v\int_0^a dA \tag{7}$$

in which a = the projected or surface area of the section swept by the flow.

Integrating:

$$q = va \tag{8}$$

Equation 8 shows that the flow of suspension down through and out of which a particle can settle is a function only of the settling rate of the particle, and of the projected area of the volume across which the considered section of flow sweeps, regardless of the direction, magnitude and variation with the depth of the flow itself.

Removals in an "Ideal" Basin

Hazen has shown that for his ideal basin, the fraction F of any particle class with settling velocity v, which will settle out of suspension and be removed, is a function of v and the "overflow" rate v_o. Thus:

When v is greater than v_o, obviously all particles settle. When v is less than v_o, the fraction settling is, from Hazen,

$$F = v/v_o \tag{9}$$

in which by definition

$$v_o = q_s/a \tag{10a}$$

in which q_s = the total flow through the section.

This same relationship holds in the less idealized flow section treated in this paper. In any section, the flow which particles of a given settling rate v can traverse is

$$q = va \tag{8}$$

and

$$v_o = q_s/a \tag{10a}$$

or

$$q_s = v_o a \tag{10b}$$

The fraction of the total section flow out of which particles of the given class will settle is obtained by dividing Eq. 8 by Eq. 10b. This is the fraction of these solids removed if they are initially uniformly distributed in the feed suspension. Thus

$$\frac{q}{q_s} = \frac{v}{v_o} = F \tag{11}$$

Since this is true for every class of particles present, it follows that re-

moval in a section is a function only of the settling rates of the particles, and of the overflow rate.

Overflow Rate and Vertical Flow Components

The overflow rate has been defined as the flow through a section, divided by the horizontal projected area across which it sweeps (Eq. 10a). Flow divided by area gives a value that has the dimensions of velocity. The total flow through the section divided by the area is numerically equal to what the average vertical component of flow velocity would be if it were assumed that the flow was rising through the sedimentation basin. This numerical equality sometimes leads to the belief that settling particles are, in actual fact, required to move against an opposing upward flow of fluid in order to settle out. Particles having a settling rate less than the vertical component of fluid flow must have a resultant upward movement, which might be expected to carry them inevitably into the overflow. This is not the case.

If the flow lines were truly vertical (Figure 2A), it is clear that a settling particle would not cross or traverse any of the stream. It must either settle down the streamline to its source, or be swept with the streamline to its destination. This would depend on whether the particle settles with respect to the fluid more or less rapidly than the fluid is moving with respect to the surroundings. However, if the flow lines are truly vertical, the flow is sweeping across no area at all, and by definition a equals zero. Equation 8 then predicts that the particle would traverse zero flow, q.

On the other hand, if the flow lines have any inclination or horizontal velocity component V at all, then any settling particle has at least some component of motion across the flow, as shown in Figure 2B. Even though

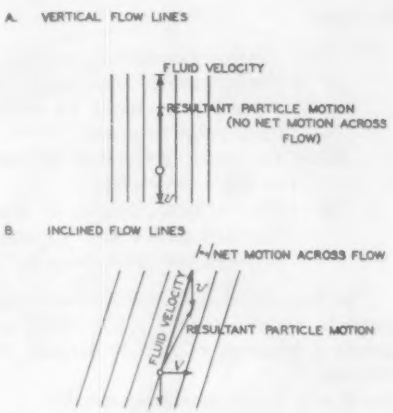

FIGURE 2.—Velocity components of a settling particle.

the particles settle exceedingly slowly and the vertical component of fluid velocity is exceptionally large, the particles must be moving from the upper to the lower strata of the flow. Hence, some particles must be accumulating on the lower boundary of the stream at the bottom of the sedimentation basin. Equation 11 shows that the number of particles settling out is independent of the vertical velocity components, provided the original assumptions are valid, and there is no scour of the deposited solids.

Effect of Density Currents

It is to be noted particularly that the removals made from suspensions having the specified particle dynamics would not be affected by: (a) so-called "density currents," which are variations of velocity (V) with depth; (b) the vertical expansion of the flow lines which sometimes takes place at the influent of a basin; and (c) the vertical contraction and updraft of the flow pattern which necessarily takes place at an effluent overflow weir. An exception to this occurs when variations in flow pattern may have secondary effects such as producing turbulence, vertical mixing, and hence departure from the assumed conditions. This

would be contrary to the findings of Ingersoll *et al.* (4) relating to the effect of the updraft at an effluent weir, unless their flow net calculations took into account such secondary effects.

While depth in itself would not affect separation according to the theoretical treatment presented above, too small a depth would lead to significant departures from the assumed flow conditions. Other things being equal, shallow depths lead to high flow velocities. Flow velocities must be kept sufficiently low to avoid scour of the deposited solids, and to avoid excessive turbulence and vertical mixing of flow paths.

The total flow handled by a sedimentation basin is the summation of the flows carried by all sections in parallel. However, if each section is to produce identical removals of particles, the overflow rate v_o or ratio of q to a must be the same for all. This condition will be fulfilled if: (a) V does not vary with w, the lateral distance measured perpendicular to the flow lines; and (b) the horizontal component x of the distance from inlet to outlet is the same measured along all flow lines. These conditions in turn will be met by most conventional types of sedimentation basins, circular or cross-fed rectangular, provided the influent is uniformly distributed laterally, and the effluent is withdrawn uniformly over the entire length of the effluent weir.

Effect of Streaming

The effect of "streaming," or short-circuiting currents resulting from non-uniform lateral distribution of flows, is dependent upon the distribution of the particle-settling velocities in the feed suspension. As a result of streaming, one section of flow is subjected to a different overflow rate than another. First, all particles having a settling rate in excess of the maximum overflow rate exhibited by any flow section in the tank will be collected, and, therefore, will not be affected by the short-circuiting.

Second, all particles having a settling rate less than the minimum overflow rate exhibited by any flow section will be collected at a rate which also is not affected by the streaming. This fact is seen to be reasonable if an ideal horizontal flow basin such as postulated by Hazen and Camp is considered. Until a certain size class of particles is depleted, the particles in this size class will continue to settle on the bottom of the sedimentation basin at a mass rate that is dependent on the particle concentration just above the bottom, and on the settling rate of the particles. Until the particles in the particular size class are depleted, the particle concentration just above the bottom remains equal to the initial concentration, since particles are settling into the lowest layer of the suspension from above as fast as they are settling out of it. Horizontal movement or flow of the suspension does not alter this settling action. Therefore, all particle classes which settle slowly enough so that they are not depleted from any of the sections of flow collect on the basin bottom at a rate which would be independent of horizontal flow and hence of streaming.

A more general mathematical proof of this proposition follows. The mass flow of a particle class with a settling rate v is

$$M_p = C_p q_s \tag{12}$$

in which M_p = mass of particle class fed through section per unit of time; C_p = concentration of particle class in feed.

The mass rate S_p at which this particle class settles out is

$$S_p = F M_p \tag{13a}$$

Substituting from Eq. 11

$$S_p = C_p q_s \frac{v}{v_o} \tag{13b}$$

then from Eq. 10a

$$S_p = C_p a v \qquad (13c)$$

Thus the mass rate at which particles of any size class, having a settling rate lower than the overflow rate, settle out of the suspension is dependent on (a) the concentration of the particle class in the feed; (b) the area through which the flow sweeps; and (c) the settling rate of the particles. It is independent of the section overflow rate provided the particle settling rate is lower than the overflow rate. Therefore, the removal rate in a sedimentation basin for particles having settling rates lower than the minimum overflow rate for any section is not affected by streaming. If the feed rate to the basin remains constant, and hence the average overflow rate of all sections, then the over-all per cent removal of these particles also remains constant.

Thus the removal of coarse particles, and of fine ones, is not affected by short-circuiting, provided always that the particles have a settling behavior as postulated. The effect of streaming on intermediate particles may be visualized as shown in the following development.

Particle Settling

It has been established that the removal from a section of flow of any class of particle would be unaffected by the geometry of the section; provided the overflow rate remains the same. Therefore, the particle removal from an ideal sedimentation basin as postulated by Hazen and Camp would be identical to that obtained in any flow section having the same overflow rate. Accordingly, an analysis of removals in Camp's ideal sedimentation basin would be valid for the less idealized basin of this paper.

In Camp's ideal sedimentation basin the direction of flow is horizontal and the velocity is the same in all parts of the settling zone. If a vertical cylinder of feed suspension, extending from the top to the bottom of the basin, were marked at the influent end, it would maintain its shape and identity as it traversed the basin (Figure 3). This is the same as though a batch settling operation was moved sideways. What happened within this cylindrical element would be indistinguishable from what would happen if settling took place batchwise, except that the settled particles would be strewn along the floor of the basin as the element moved along, instead of accumulating in a pile. A relationship between the removal of solids and their settling velocity distribution in batch settling has been presented by Oden (5). Appropriately translated, Oden's analysis would be valid for Camp's ideal sedimentation basin, and hence for any flow section as previously defined.

Figure 4 shows the weight of solids

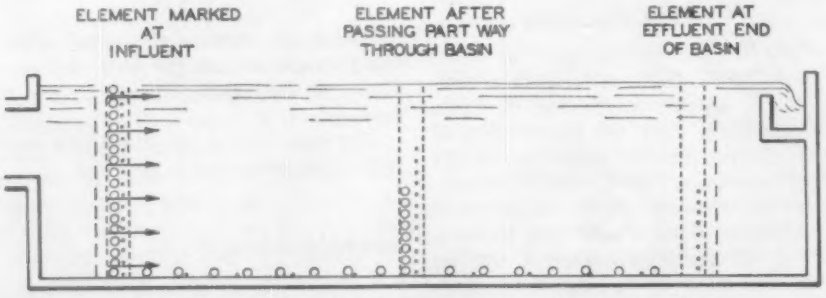

FIGURE 3.—Relationship between continuous and batch settling.

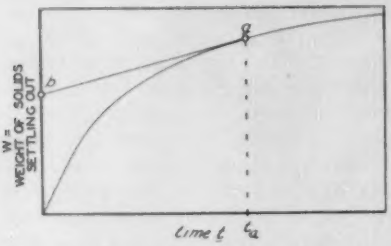

FIGURE 4.—Oden settling curve.

w settling out of a cylinder of suspension of height h and accumulating on the bottom, as a function of time t. At any point a on the curve, particles having a settling velocity of h/t_a will just be depleted. All of the faster settling particles will have settled out already. Slower settling particles will be settling out at a rate equal to the slope of the curve at point a, or $(dw/dt)_a$. They had been settling out at this same rate since the start of the operation, because originally they were uniformly dispersed in the suspension, and each particle settled with a constant velocity. The curve will have a decreasing slope with increasing time as a result of successive depletion of particle size classes with ever decreasing settling velocities.

The weight of particles having a settling velocity lower than h/t_a will be equal to the weight of the particles which have settled out up to time t_a, plus the weight of solids remaining in suspension after time t_a. Mathematically, the weight of the slower settling particles that have settled out up to time t_a is equal to $(dw/dt)_a\, t_a$. Therefore, the weight of the particles originally present, having settling velocities less than h/t_a, is $(dw/dt)_a\, t_a + (w_\infty - w_a)$. The term w_∞ is the weight of the particles which would settle out after an infinite time, or the weight of the particles originally present. The weight of the particles having a particle-settling velocity in excess of h/t_a, therefore, will be equal to

$$w_\infty - \left(\frac{dw}{dt}\right)_a t_a - (w_\infty - w_a) \quad (14a)$$

or

$$= w_a - \left(\frac{dw}{dt}\right)_a t_a \quad (14b)$$

Equation 14b is recognizable as the value of w at the intercept of the tangent drawn to the curve at a, with the vertical axis w. This is Oden's method for determining the settling velocity distribution (and with the aid of Stokes' Law the size distribution), from a plot of the weight of the particles settling out versus time.

If the vertical scale of Figure 4 was divided by w_∞ and the horizontal scale by h, the shape of the curve would not be changed, but only the modules of the axis scales. Oden's tangent construction would still be valid. However, the vertical scale would then measure fractional removal, and this could be indicated as a per cent fraction. The horizontal scale t/h becomes equal to l/v_o for the ideal sedimentation basin. Figure 5 shows the scales modified accordingly. The tangent intercept at b indicates the per cent of the solids originally present which had a settling velocity in excess of the value of v_o at the tangent point a.

Hypothetical Case

The effect of streaming can be visualized by assuming that one-half the flow to a sedimentation basin streams

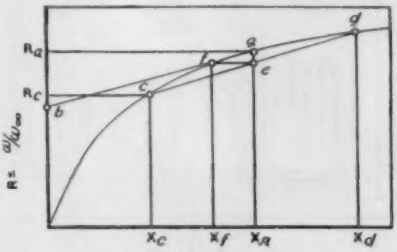

FIGURE 5.—Overflow rate.

in a manner that subjects it to greater than the average overflow rate for the basin (Figure 6). Assume that point a on Figure 5 corresponds to the average overflow rate for the basin. Then point c corresponds to the overflow rate of the short-circuiting stream, and R_c is the removal to be expected from the half of the flow thus streaming. Observing that the abscissa of Figure 5 is l/v_o or a/q, it is apparent that its value is proportional to the area a, if the values of q are equal. Thus what the streaming half loses, the other half gains. Therefore, point d, corresponding to the overflow rate of the residual half of the flow, is as far to the right of point a as point c is to the left. The average removal for the two half sections of the flow would then be represented by point e, the intersection of a straight line between points c and d, with a vertical line at t_a. The loss in recovery is represented by the distance e to a. The same recovery could have been obtained in a sedimentation basin without streaming and an overflow rate corresponding to point f. Since the total flow to be handled remains constant, the area required in the non-streaming basin would be X_e/X_f or X_a/X_f times the area required in the one streaming as assumed. Thus the areal efficiency E of the streaming basin can be designated as X_f/X_a.

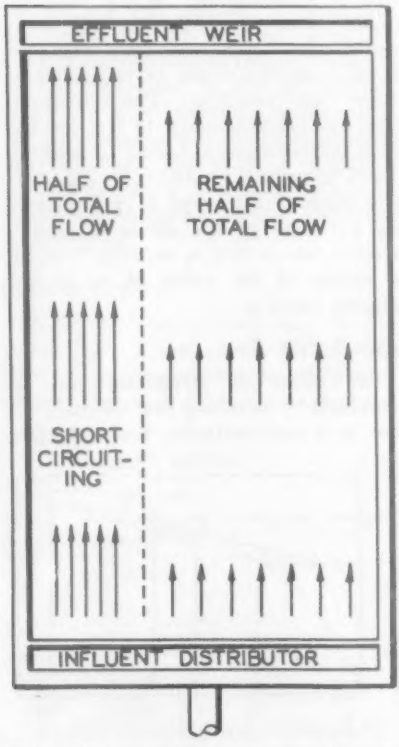

FIGURE 6.—Hypothetical streaming basin.

Radial Flow vs. Long, Rectangular Tanks

Streaming, or the lack of lateral distribution of the flow, is not difficult to avoid with properly designed influent and effluent weir structures. The drag on the side walls or longitudinal baffles might be expected to minimize severe streaming resulting from grossly improper introduction of the feed. However, the drag on such members also could be expected to create an uneven lateral distribution of flow as shown by Camp (2). Hubbell (6) remarks on the relative ease of getting good lateral distribution in a siphon-fed radial basin.

It is clear that dye tests, which do not distinguish between density current short-circuiting and streaming, cannot be a valid general criterion of the behavior of a sedimentation basin. This has been pointed out by Eliassen (7). Therefore, the fact that a long narrow rectangular sedimentation basin shows less longitudinal dispersion of dye than conventional basins cannot, as tacitly assumed by Camp, be taken as a theoretical indication of their superiority for sedimentation. Theoretically, the long sedimentation basins may even be inferior, in that they minimize harmless density current short-circuiting at the expense of introducing harmful streaming as a result of side-wall drag.

The second criterion advanced for the superiority of long narrow sedimentation basins is that (in short flow line units) "the size of the effective settling zone is reduced because the inlet and outlet zones occupy too great a part of the total path of flow." (3) It is assumed these inlet and outlet zones each have a flow path length equal to the depth of the sedimentation basin, and that negligible settling takes place in these zones. Theoretically, except for turbulence effects the distortion of the flow net at the inlet and outlet ends of a sedimentation basin has been shown in this paper not to affect the sedimentation of particles having the settling dynamics postulated by Hazen and Camp.

Since both of the principal theoretical criteria relied on are of questionable validity, the conclusion (3) "that long, narrow, rectangular tanks are vastly superior in performance to radial flow tanks" is seen to lack general theoretical validation. It cannot be accepted as a general conclusion.

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BAVARIAN BIOLOGICAL INSTITUTE, MUNICH, GERMANY

An introductory course on the special problems of biology and sewage and stream sanitation will be presented March 5–9, 1956, at the Bavarian Biological Institute, Munich, Germany. The studies are under the direction of Dr. R. Demoll and Dr. H. Liebmann. An advance course on the biological and chemical problems related to the utilization of fluid and solid wastes will be given at the Institute on October 8–12, 1956.

Reservations must be received by February 25 for the spring course, and by September 22 for the fall term. Fees are 45,-DM ($10) and 50,-DM ($11), respectively. Information may be obtained from Dr. H. Liebmann, Bayerische Biologische Versuchsanstalt, Munchen 22, Veterinärstrasse 13.

SYNTHETIC DETERGENTS AND THE B.O.D. TEST *

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The increasing use of synthetic detergents in industry and in the home has created many new problems for the sewage treatment plant operator. Numerous investigations have been carried out to determine the influence of these materials upon various steps in the sewage treatment process. The test procedure commonly involves the addition of a detergent to the influent of the particular treatment process, followed by a determination of the efficiency of B.O.D. removal in the presence of the detergent. This procedure introduces a second variable, a fact which has usually been overlooked or not considered significant by the investigators. The new variable develops in the following way. It is known that the detergent is seldom removed entirely in any sewage treatment process; therefore, the effluent will contain a certain proportion of the original dosage of detergent. The second variable, then, is the effect of this detergent residual on the B.O.D. of the effluent.

This paper is a report of studies on the influence of selected synthetic detergents, surface-active agents and detergent builders on the standard dilution B.O.D. test. Only a few scattered references to the effect of syndets on the B.O.D. were available in the literature until Bogan and Sawyer (1) published their paper on the biological degradation of synthetic detergents. A general survey of the characteristics and effects of synthetic detergents was presented in the report (2) of the Task Group on Synthetic Detergents in Water Supplies of the American Water Works Association.

Materials and Methods

Since domestic sewage contains unknown quantities of various detergents, it was necessary to employ a synthetic sewage. Difco Lactose Broth was used as the synthetic sewage in this study. The stock medium was prepared and sterilized according to the manufacturer's directions. A working solution of this medium was prepared fresh each day by diluting 5 ml. of the sterile stock to 125 ml. with distilled water. This working solution had a 5-day B.O.D. of approximately 350 p.p.m.

The synthetic sewage was seeded with settled sewage obtained from the Waste Treatment Laboratory at the Ohio State University. One milliliter of seed (settled sewage) was added to each 100 ml. of the working solution.

Detergents Studied

Various syndets and surface active materials, ranging from commercial products to pure surface-active agents, were investigated. In addition, several of the common detergent builders also were checked for effect on the B.O.D. test. The following list of the materials examined includes a brief description of the chemical composition of each surfactant as obtained from the "Encyclopedia of Surface-Active Agents" (3). The content of active agent is given when this value is known. In the case of the household detergents this content is understood

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to be in the range of 30 to 35 per cent.

Anionic type

1. Sodium lauryl sulfate, C.P.
2. Igepon T, 33 per cent active agent.
   Composition—sodium oleyl methyl tauride.
3. Cyclopon A
   Composition—sulfonated amide of amino alcohol.
4. Santomerse No. 1
   Composition—sodium dodecyl benzene sulfonate.
5. Nekal BX, 68 per cent active agent.
   Composition—mixture of sodium dibutyl naphthalene sulfonate and sodium diisopropyl naphthalene sulfonate.

Cationic type

1. Cetyl trimethyl ammonium bromide (CTAB), C. P.
2. Cetyl pyridinium chloride (CPC), C.P.

Nonionic type

1. Igepal CA Extra High Concentration, 100 per cent active agent.
   Composition—condensation product of a dodecyl phenol with 12 molecules of ethylene oxide.
2. Tween 80, 100 per cent active agent.
   Composition—condensation product of sorbitan monooleate and ethylene oxide.

Detergent builders

1. Sodium sulfate, C.P.
2. Tetrasodium pyrophosphate, C.P.

Commercial household detergents

1. Tide, anionic type.
2. Surf, anionic type.
3. Fab, anionic type.
4. All, nonionic type.
5. Thermo-San, detergent-sanitizer containing 10 per cent alkyl dimethyl benzyl ammonium chloride (cationic) and 18 per cent alkyl aryl polyether alcohol nonionic).

Laboratory Technique

Standard solutions were prepared on the basis of detergent weight rather than on the weight of content of surface-active agent. These solutions were kept in the refrigerator to inhibit growth of microorganisms, which occurs readily at room temperature in the case of certain detergents such as Tween 80 and Igepon T.

The technique for the standard dilution B.O.D. test (4) was used in this study. The dilutions were prepared in a specially-designed graduated glass cylinder equipped with a convenient draining tube. Exactly 10 ml. of seeded working solution of the synthetic sewage was added to a small volume of dilution water in the graduated cylinder, followed by the desired volume of detergent solution. The volume was then made up to 1000 ml. and, after thorough mixing, two water-seal D.O. bottles were filled. One bottle was used for the initial D.O. determination, while the second bottle was incubated at 20° C. for five days. An initial D.O. determination was made on each dilution in order to avoid errors resulting from oxygen entrained during the mixing and also to detect any iodine demand of the detergent. Controls were set up in the same way, but without the addition of detergent.

The following concentrations of the test materials were studied: 0.1, 0.2, 0.4, 0.5, 0.6, 0.8, 1.0, 1.5, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 50, 80, and 100 p.p.m. It must be pointed out that these concentrations represent the content of detergent after the synthetic sewage was diluted sufficiently to obtain 30 to 70 per cent oxygen depletion in five days.

Experimental Results

The cationic detergents showed a definite iodine demand, as indicated

TABLE I.—Iodine Demand of Cetyl Trimethyl Ammonium Chloride

Conc. of CTAB (p.p.m.)	Determination (p.p.m.)	
	Initial D.O.	Iodine Demand
0	6.70	—
10	5.80	0.90
20	5.20	1.50
50	3.00	3.70
80	2.80	3.90
100	2.20	4.50

TABLE II.—Iodine Demand of Cetyl Pyridinium Chloride

Conc. of CPC (p.p.m.)	Determination (p.p.m.)	
	Initial D.O.	Iodine Demand
0	6.50	—
10	5.00	0.50
20	6.00	0.50
50	4.20	2.30
80	2.20	4.30
100	2.00	4.50

by a low initial dissolved oxygen compared with the dissolved oxygen of the control. Tables I and II show that both CTAB and CPC exhibit an iodine demand which increases markedly with increasing syndet concentration. There was no appreciable iodine demand at concentrations below 10 p.p.m. None of the other test materials showed any evidence of iodine demand in the concentration range studied.

Certain of the detergents interfered with the conventional Winkler D.O. test, specifically, in the titration with sodium thiosulfate. When Santomerse, Tide, Fab, Tween 80 and CTAB were present in concentrations greater than 10 p.p.m., it proved very difficult to remove the last trace of the blue color of the starch-iodine complex; consequently, the danger of overtitration was introduced. In the presence of Igepal and Tween 80, addition of the D.O. reagents resulted in the formation of a heavy, brick-red precipitate. This phenomenon has been reported previously in the case of Igepal by Gloyna (5). The precipitate disappeared during titration and did not interfere with the endpoint; however, it was necessary to hit the endpoint quickly since there was a rapid return of the blue indicator

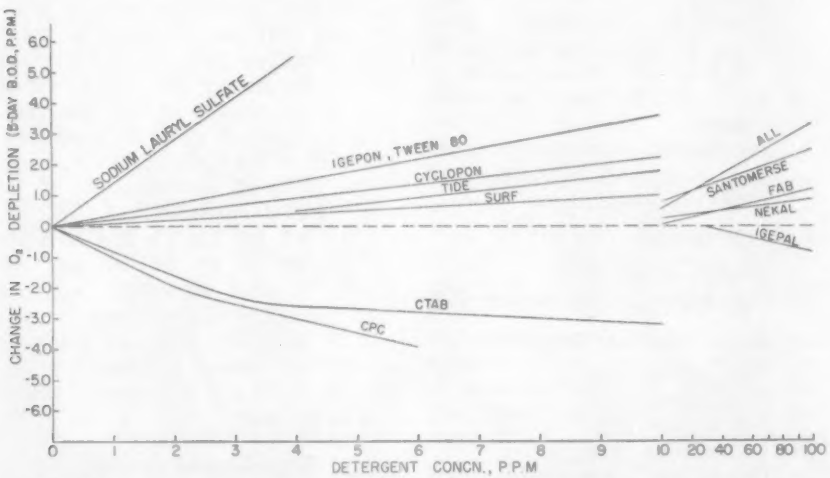

FIGURE 1.—Oxygen depletion resulting from the addition of syndets to a seeded synthetic sewage.

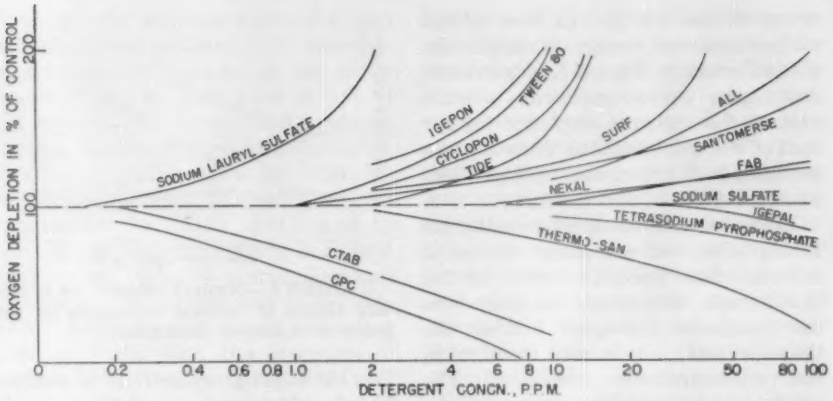

FIGURE 2.—Per cent oxygen depletion due to syndets compared with depletion of control.

color. Failure to complete the titration promptly, led to over-titration.

Oxygen Depletion

The influence of the test materials on the B.O.D. of the seeded synthetic sewage is shown graphically in Figures 1 and 2. Each curve represents the average of two to four individual runs. In Figure 1 the data are plotted as an increase (positive values) or decrease (negative values) in oxygen depletion resulting from added increments of detergent. These curves represent absolute values (the B.O.D. of the detergent). The values for the detergent builders are not included in this graph. Since, in most cases, the change in oxygen depletion is a straight-line function of the concentration of the detergent, the rate of change is uniform and may be calculated from the slope of the curve.

$$m = \frac{\Delta y}{\Delta x}$$

$$= \frac{\text{Change in oxygen depletion (p.p.m.)}}{\text{Detergent concentration (p.p.m.)}}$$

These values are listed in Table III. The B.O.D. is obtained from the rate of change by multiplying by a factor of 1,000,000. The curves for CTAB and CPC show a changing slope. The first part of each curve, however, appears to be linear and the values recorded are obtained from that portion of the curve between 0 and 2 p.p.m.

The practical significance of the results is more apparent in Figure 2 in which the data are plotted as increased or decreased oxygen depletion based on the depletion of the control as 100 per cent. Considering 100 per cent depletion of the control as the base line, a curve with a positive slope represents an increase in oxygen depletion or an increase in the apparent B.O.D. of the sewage. Conversely, a curve with a negative slope indicates a de-

TABLE III.—Rate of Change in Oxygen Depletion and B.O.D. of Synthetic Detergents

Detergent	$\frac{\Delta y}{\Delta x}$	5-Day B.O.D. (p.p.m.)
Sodium lauryl sulfate	1.38	1,380,000
Igepon T	0.36	360,000
Cyclopon A	0.23	230,000
Santomerse No. 1	0.02	20,000
Nekal BX	0.01	10,000
Tide	0.12	120,000
Surf	0.10	100,000
Fab	0.01	10,000
Tween 80	0.36	360,000
All	0.03	30,000
Igepal CA	−0.01	—
CTAB	−0.80	—
CPC	−0.95	—

crease in the B.O.D. In view of the uniform rate of change in oxygen depletion noted in Figure 1, it is evident that these curves represent relative values, since oxygen depletion in per cent of control, resulting from a given increment of detergent, is dependent on the B.O.D. of the control.

Each of the detergent materials tested, with one exception, showed a definite effect upon the value of the B.O.D. test. The single exception was the inorganic detergent builder, sodium sulfate, which was inactive in the concentrations examined. The group producing an increase in the B.O.D. was composed principally of the anionic detergents, although included were the nonionic syndets, Tween 80 and All. The relative effects of these substances in influencing the B.O.D. formed a broad spectrum with sodium lauryl sulfate the most active and Nekal the least active.

Cationic Detergents

The cationic detergents had a marked inhibitory effect on the B.O.D. of the synthetic sewage. The nonionic syndet, Igepal, and the detergent builder, tetrasodium pyrophosphate, were also somewhat inhibitory in the higher concentrations.

It is frequently stated that the inhibitory effects of cationic detergents are completely neutralized by the presence of anionic syndets. Experiments were carried out to determine whether such neutralization occurred in the B.O.D. test. Results of these experiments are presented graphically in Figure 3. The point at which the curves for the anionic-cationic combinations intersect the base line (100 per cent) represents complete neutralization of the inhibitory effect of the cationic detergents. The data indicate complete neutralization of the inhibitory activity of 3 p.p.m. CTAB or CPC by sodium lauryl sulfate in a concentration between 1 and 3 p.p.m. It is also apparent from the graph

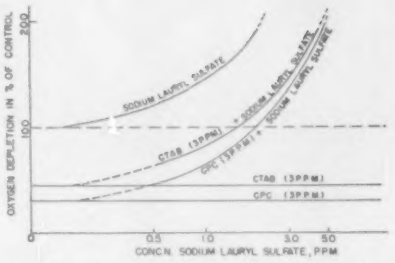

FIGURE 3.—Neutralization of the inhibitory effects of cationic detergents by the presence of anionic detergents.

that the stimulatory activity of sodium lauryl sulfate was partially neutralized by the cationic syndets. Nekal had an effect similar to that of sodium lauryl sulfate, although larger concentrations of this detergent were required to reverse the inhibition. The activity of Thermo-San suggests that the nonionic syndets do not exert a neutralizing effect on the cationic detergents.

Discussion

It is evident that individual synthetic detergents show characteristic effects on the B.O.D. test. These effects may assume considerable significance under certain circumstances. The fact that cationic detergents have an iodine demand could introduce a serious source of error in the B.O.D. determinations of those laboratories which do not routinely run an initial D.O. determination. Failure to do this would lead to high B.O.D. values. Unfortunately these studies do not show whether or not the iodine demand of the cationic types is neutralized by an excess of anionic detergent. Interference with the titration endpoint, while not a serious problem, could conceivably cause the technician some difficulty and might introduce a small error in the B.O.D. determinations.

The data emphasize the fact that the presence of extremely small concentrations of detergents may have a considerable effect on the B.O.D. of the sewage. This was particularly

true for the cationic detergents and sodium lauryl sulfate. For example, approximately 2 p.p.m. of the latter was sufficient to double the B.O.D. of the sewage. The commercial products, Igepon, Cyclopon, Tide and Tween 80, were also quite active in relatively low concentrations, producing sizeable increases in the B.O.D. For this group of substances, their presence in concentrations of 10 to 15 p.p.m. increased the B.O.D. to 200 per cent of the control. Although less active, the other test materials, with the exception of sodium sulfate, altered the B.O.D. of the sewage to some degree. It must be emphasized that these concentrations represent the detergent content after the sewage has been diluted sufficiently to obtain a 30 to 70 per cent D.O. depletion. Thus, 2 p.p.m. of detergent in a one per cent sewage dilution would mean an initial concentration of 200 p.p.m., whereas 2 p.p.m. of detergent in a ten per cent dilution of sewage would represent an initial concentration of 20 p.p.m. in the undiluted sewage.

Effect of Chemical Composition

It is interesting to attempt to correlate the observed effects with the chemical composition of the active agent of the anionic detergent, although no general conclusion can be reached on the basis of this limited number of compounds. In addition, the concentration of surfactant in the detergent is a variable factor. However, taking into consideration the information available, it is possible to arrange the detergents in an activity spectrum as follows:

Sodium lauryl sulfate > Igepon > Cyclopon > Santomerse > Nekal, where sodium lauryl sulfate is most active in increasing the apparent B.O.D. of the sewage and Nekal the least active, with the other detergents intermediate in activity. Sodium lauryl sulfate is a straight chain compound, as are the surfactants in Igepon and Cyclopon. In Santomerse and Nekal the active agents are cyclic compounds. These findings confirm the conclusions of Bogan and Sawyer that the sewage organisms metabolize long chain surfactants more readily than the ring structures. Nekal, a naphthalene compound, is apparently less susceptible to the metabolic activities of the microorganisms than is the benzene derivative, Santomerse. It is possible that the organisms are able to attack only the alkyl substituents of these ring compounds. The different activities of the two compounds could arise from the relative lengths of the alkyl chains, short in the case of Nekal (isopropyl and isobutyl) and long in the case of Santomerse (dodecyl).

Inhibitory Effects

As anticipated, the presence of cationic detergents produced a diminished uptake of oxygen in the B.O.D. test. These agents were active in very low concentrations. Both CTAB and CPC in concentrations of 2 p.p.m. were able to reduce the B.O.D. of the sewage by 50 per cent. At concentrations of 10 p.p.m. the sewage showed essentially no oxygen depletion. Since it has been well established that cationic detergents are active bactericidal and bacteriostatic agents, a reduction in apparent B.O.D. as a result of their inhibitory action on the sewage organisms was to be expected.

As demonstrated by this and other studies, the inhibitory effects of the cationic detergents are neutralized by the anionic syndets. Since the anionic syndets predominate in domestic sewage at the present time, it is not likely that the cationic agents, which are used principally as sanitizers rather than cleansers, will create an immediate problem in sewage treatment. However, under certain circumstances their activity might be important—for example, if the nonionic detergents replace the anionic types in consumer preference. Another specialized case

would be that of a milk processing plant which maintained its own waste treatment system and which used large quantities of cationic detergents for sanitizing purposes. In both cases the usual excess of neutralizing anions would be absent and the cationic syndets would be free to exert their normal effects. The result would be low B.O.D. values and an over optimistic evaluation of waste treatment efficiency.

Apparently the nonionic detergents may stimulate or inhibit the growth of the microorganisms in the B.O.D. test, depending on the chemical formulation of the surfactant molecule. It may be postulated that the inhibitory activity of Igepal is a result of the phenolic configuration. On the other hand, Tween 80 is a cyclic dehydration product of sorbitol, a sugar alcohol readily utilized as an energy source by a wide variety of microorganisms. Accordingly, it is not surprising that this surfactant increases the B.O.D.

Of the two detergent builders, sodium sulfate was inactve while tetrasodium pyrophosphate was inhibitory at concentrations of 10 p.p.m. and higher. The latter is a sequestering agent, capable of tying up calcium and magnesium ions in a soluble complex. Many organisms have a nutritional requirement for these ions; hence, it is possible that the inhibitory activity of the compound rests in its ability to reduce the number of available calcium or magnesium ions below the level necessary for bacterial growth and metabolism. Since the major proportion (60 to 70 per cent) of many synthetic detergents consists of builder materials, the fact that certain of these detergent builders may affect the natural microflora of sewage is considered significant.

Conclusion

It seems obvious that the preceding effects on the B.O.D. test cannot be ignored in the interpretation of experimental results of investigations on the influence of detergents on sewage treatment processes. The point is best discussed by considering a hypothetical investigation. The researcher was investigating the effect of an alkylaryl sulfonate detergent in concentrations of 10 to 100 p.p.m. on the efficiency of a trickling filter. Definite increases in the B.O.D. of the filter effluent were recorded compared with the B.O.D. of the effluent in the absence of the detergent. These increases were interpreted as a deterioration in filter efficiency. It was known that the concentration of detergent was reduced 50 per cent by passing through the filter. The detergent was present, however, in considerable quantity in the effluent and was capable itself of increasing the B.O.D. In view of these facts, the following question must be considered: was the increase in B.O.D. the result of decreased efficiency of the filter in the presence of the detergent or was it the result of the presence of the detergent in the test sample? The answer is that probably both factors influenced the effluent B.O.D.

The true effect of the detergent on the efficiency of the filter can be ascertained by correcting for the increase in B.O.D. resulting from the presence of detergent in the effluent. This is readily accomplished by determining the rate of change of the B.O.D. as produced by the test detergent. The rate of change is multiplied by the concentration of detergent in the effluent and this value is then subtracted from the experimentally-determined B.O.D. value of the effluent. This corrected effluent B.O.D. value may then be compared with the B.O.D. value of the influent in order to obtain a true picture of the efficiency of B.O.D. removal in the presence of the detergent.

Summary

An investigation of the effects of selected synthetic detergents, surface-

active agents and detergent builders revealed the following:

1. The cationic detergents, cetyl trimethyl ammonium bromide and cetyl pyridinium chloride, exhibited an iodine demand in concentrations greater than 10 p.p.m.

2. Santomerse, Tide, Fab, Tween 80, CTAB and Igepal interfered with the D.O. titration.

3. All the anionic detergents tested increased the apparent B.O.D. value of the synthetic sewage, although the individual detergents varied considerably as to the magnitude of the increase. It is suggested that this activity of the detergent is a function of the susceptibility of the chemical structure of the surfactant molecule to metabolization by the sewage organisms.

4. Of the two nonionic detergents tested, Tween 80 in concentrations greater than 1 p.p.m. was stimulatory to the microorganisms in the B.O.D. test, while Igepal was inhibitory in concentrations greater than 30 p.p.m.

5. The cationic detergents decreased the apparent B.O.D. value of the synthetic sewage.

6. An excess of anionic detergent neutralized the inhibitory activity of the cationic syndets.

7. One of the detergent builders, tetrasodium pyrophosphate, was inhibitory to the microorganisms in the B.O.D. test in concentrations of 10 p.p.m. and greater.

The experimental results emphasize the necessity of correcting for the effects of synthetic detergents on the B.O.D. test in any study of the influence of detergents on a specific sewage treatment process.

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PROFESSIONAL OPPORTUNITY

The Florida Merit System is accepting applications from sanitary and public health engineers for positions with the state and local health departments. Positions are available for beginning and experienced engineers with college degrees.

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WESTCHESTER COUNTY SOLVES SOME SEWERAGE PROBLEMS *

By Guy Griffin and James Brown

Deputy Commissioner and Superintendent, New Rochelle Sewage Treatment Plant, Respectively, Department of Public Works, Westchester County, N. Y.

Historical Background

The New York State Legislature, in 1895, created a Bronx Valley Sewer Commission to study the expediency of constructing a sewer through that portion of the City of New York along the Bronx River, now the Bronx and Westchester County. The study was financed jointly by New York City and Westchester County.

The engineers employed at that time studied the possibility of providing one large or several small sewage treatment plants to abate pollution of the Bronx River. Results of the survey favored a trunk sewer to receive sewage from both the Bronx and Hutchinson River Valleys and discharge it to deep water in Long Island Sound near either Hart Island, High Island or City Island (Figure 1). The engineers advised that the sewage be held in tanks during flood tide and discharged on the ebb tide. Furthermore, provision should be made for the separation of solid matter from the liquid. Thus, early consideration was given to the provision of primary sewage treatment.

The Bronx River sewer was constructed in 1911 only to the vicinity of the New York City boundary where it turned westward through a tunnel and discharged into the Hudson River instead of Long Island Sound. Later, in 1931, the present screening plant in South Yonkers was constructed at the tunnel portal astride the twin 48-in. cast iron outlets, which are still in use for the plant outfall. At about this same time, sewage from the Hutchinson River Valley was added by means of a tunnel and sewer line to the Bronx Valley sewer. Engineering foresight had provided for the possibility of this additional sewage flow by adequately sizing the original Bronx River sewer tunnel.

In the period 1926 to 1933 the Westchester County Sanitary Sewer Commission was organized and undertook a program of planning, design and construction. Work was directed toward eliminating all objectionable pollution from both Long Island Sound and the Hudson River that could be caused by wastes from Westchester County. This program provided for sewer districts conforming generally to drainage areas. These areas were served by trunk sewers discharging through screening and disinfecting plants to submerged outfalls in the deep water of Long Island Sound and the Hudson River. The outfalls vary in length from several hundred feet in the Hudson River to approximately two miles at the Mamaroneck Sewage Treatment Plant on Long Island Sound. The screens at each of these plants are of the Reinsch Wurl type, consisting of rotating discs with slotted plates cleaned by revolving brushes.

Sewer Districts

Figure 2 shows, schematically, the sewer districts, trunk sewers, location

* Presented at 28th Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Atlantic City, N. J.; Oct. 10–13, 1955.

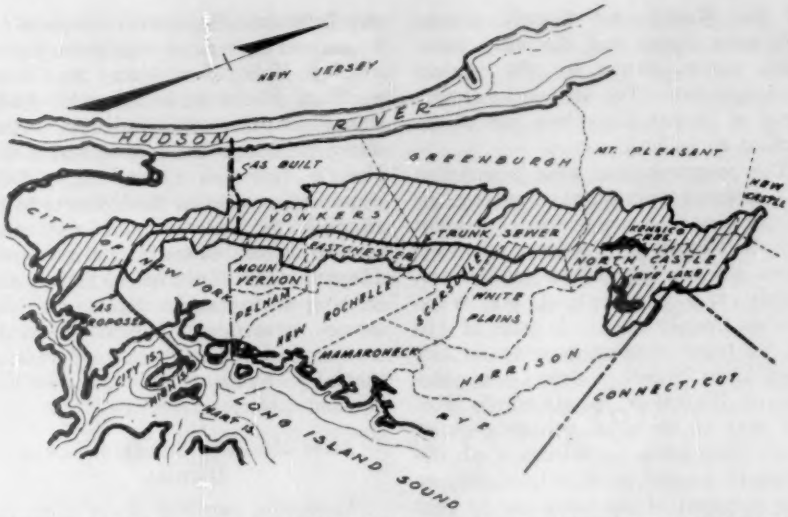

FIGURE 1.—Originally proposed Bronx Valley trunk sewer, Westchester County, N. Y.

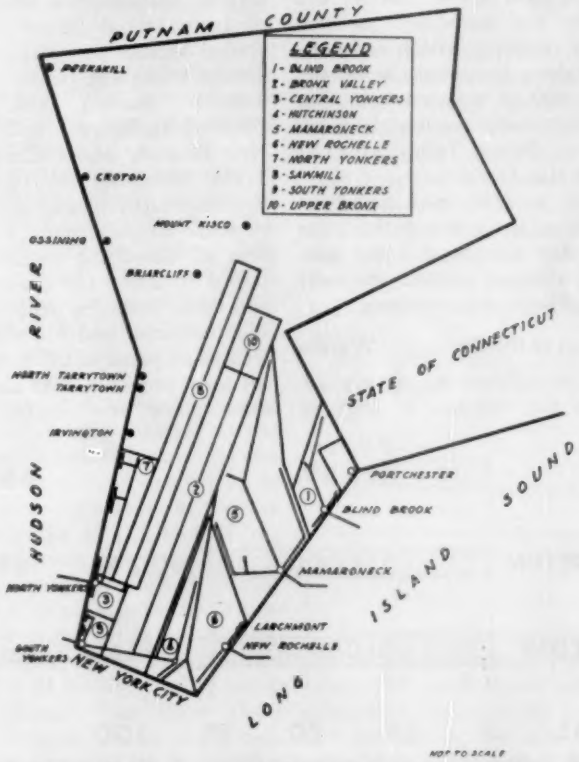

FIGURE 2.—Existing sewer districts, Westchester County, N. Y.

of the Westchester County sewage treatment plants and the other treatment plants owned by the various municipalities. The sewage treatment plant at Mount Kisco was one of the earliest to be built.

The proportions of area, population and assessed valuation served by the existing facilities are presented graphically in Figure 3. Ten sewer districts cover 130 sq. mi., which is approximately 28.5 per cent of the Westchester County area. A total of 115 mi. of trunk sewers, ranging in size from 12 to 78 inches, serves a population of 514,000 or approximately 80.3 per cent of the total county population. The assessed valuation of the property served is $1,801,000,000, or 89.8 per cent of the total county real estate valuation. Capital and operating costs in each sewer district are spread over the equalized assessed valuation of property within each district. Therefore, the county sewer tax rate per $1,000 of assessed valuation varies in the various districts from $0.63 for the Bronx Valley to $3.31 for the Saw Mill District.

From 1933 to 1950 there had been little change in the sewerage facilities of Westchester County but the continued population growth brought changes in effluent requirements.

Classification of the Receiving Waters

Changes in effluent standards resulted from the creation in 1936 of the Interstate Sanitation Commission by mutual agreement among the States of New York, New Jersey and Connecticut. Following considerable study this commission classified the waters along the Long Island shore and in the Hudson River as Class "A." This classification defined these waters to be suitable for recreational use and standards of effluent quality were adopted which could not be met by the existing county and several municipal sewage treatment plants. Several municipalities then discharged raw or partially chlorinated sewage into the Hudson River.

New Rochelle Sanitary Sewer District

When the original sewer districts of Westchester County were formed, several communities along the shore of Long Island Sound were not included because they either already had sewage treatment plants or had other reasons. Among these were Port Chester, Larchmont and portions of New Rochelle and Pelham Manor.

The Interstate Sanitation Commission instructed the city of New Rochelle to abate its pollution. Accordingly a firm of consulting engineers was retained to study the sewage treatment and water pollution problem. A report was presented and plans for a sewage treatment plant and intercepting sewers were prepared. At about the time these plans were ready (1949), the

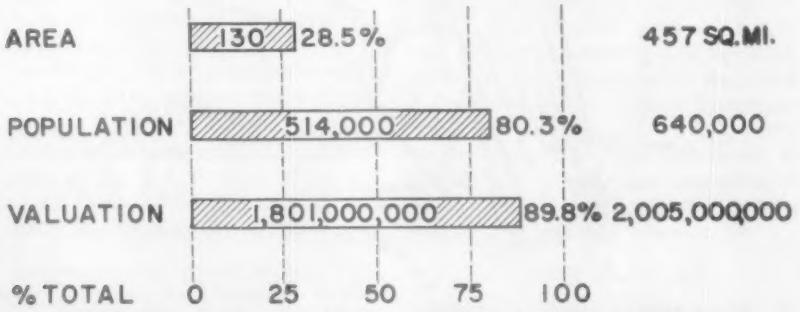

FIGURE 3.—Percentage of Westchester County, N. Y., served by the county sewer system.

Westchester County Board of Supervisors authorized a more extensive study to include Larchmont, New Rochelle and Pelham Manor in a new sewer district. This study resulted in the formation of the New Rochelle Sanitary Sewer District. County plans included a sewage treatment plant to be located in New Rochelle; an intercepting sewer from Larchmont together with a new pumping station in Larchmont; an intercepting sewer from Pelham Manor, including two siphon sections and serving part of New Rochelle; and a third interceptor and siphons to receive sewage from the vicinity of the existing New Rochelle sewage treatment facility.

Interceptor Sewers

Specifically the interceptor sewer system consists of the following facilities:

1. A 9,000 ft. 54-in. reinforced concrete outfall sewer that terminates in 40 ft. of water at the edge of a descent to greater depths.

2. The Easterly or Larchmont Intercepting Sewer which consists of 2,000 ft. of 27-in. and 3,100 ft. of 30-in. reinforced concrete pipe. This sewer line crosses Premium River, passing through the marshland and along the shore of Echo Bay, to the new sewage treatment plant in New Rochelle.

3. An unattended sewage pumping station in Larchmont located on the shore of Long Island Sound. This station will receive a large proportion of the sewage from Larchmont and pump it over the ridge to the end of the gravity Larchmont Interceptor.

4. The Westerly or Pelham Manor Interceptor which starts at the Whitewood Avenue pumping station, but has an 18-in. branch sewer from Pelham Manor and a 24 to 36-in. relief sewer line from Mount Tom Road along Whitewood Avenue. From the Whitewood Avenue pumping station a 42-in. gravity sewer line will extend along the shore of Long Island Sound to the foot of Drake Ave. on New Rochelle Creek. Siphons will carry the sewage under this creek to the southwest end of Davenport Neck and then on grade along the north edge of Davenport Neck to Hudson Park where it will enter a new 400-ft. tunnel in rock to intersect the existing 350-ft. sewer tunnel under Hudson Park. Another pair of siphons under Echo Bay will terminate at a metering chamber on the sewage treatment plant site. An interesting feature of the existing tunnels is the reversal of slope and direction of flow created by the new treatment plant site.

5. A third pair of siphons under Stephenson Run will convey the sewage to be diverted from the existing sewage treatment plant.

Costs

With the completion of the interceptor sewer project, all the sewage which previously entered Long Island Sound between New York City and the state of Connecticut, except for that from the Village of Port Chester, will have been intercepted and treated at existing sewage treatment plants or the new treatment plant in New Rochelle. The Village of Port Chester has its own primary type sewage treatment plant.

An appropriation of $8,980,000 has been made for the entire New Rochelle Project as follows:

Outfall Sewer	$1,023,000
Treatment Plant:	
Foundation, Sedimentation Tanks, and Digesters	2,075,000
Superstructures and Equipment	1,840,000
Intercepting Sewers & Pumping Stations	4,042,000
Total	8,980,000

The aerial photograph (Figure 4) shows the new sewage treatment plant site under construction and the several interceptor and outfall lines (dotted). At the top is the new 54-in. outfall sewer to Long Island Sound. Just below it is the old, shorter outfall from

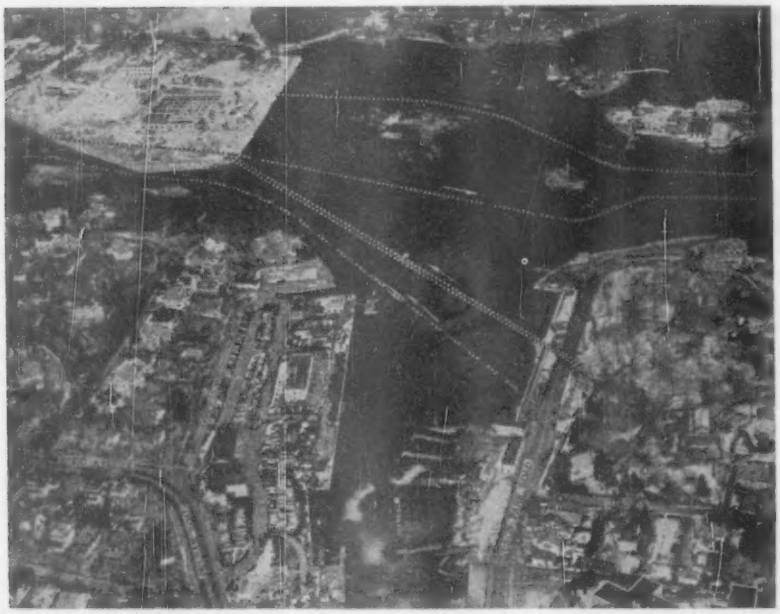

FIGURE 4.—Old and new sewer lines in Echo Bay, site of the New Rochelle primary type sewage treatment plant.

the existing screening facilities. These old facilities are located to the left, out of the photograph. The double line represents the 24- and 30-in. siphons of the main interceptor. Not shown is the Easterly Interceptor and the Stephenson Run Siphon from the old plant.

Proposed County-Wide Program

On August 28, 1952, the Interstate Sanitation Commission issued an order concerning the discharge of sewage or other polluting matters from Westchester County treatment facilities into the waters of Long Island Sound and the Hudson River. This order provided a schedule for studies and reports on all the county sewage treatment plants and, in addition, set dates for the completion of plans and completion of construction of the improved facilities discharging into the Hudson River.

A few of the highlights of this extensive county-wide project will be given here.

The Interstate Sanitation Commission order stipulates treatment to remove all floating solids and at least 60 per cent of the suspended solids. A reduction of coliform organisms shall be effected so that the probable number of such organisms shall not exceed one per cubic centimeter in more than 50 per cent of the samples of effluent tested by the partially confirmed test. This order further specifies that for the Hudson River new facilities to meet these requirements shall be in operation by December 31, 1958.

A consulting engineering firm was retained by Westchester County to report on the best solution of the problem. Their studies were to:

1. Determine the type and size of the necessary sewage treatment works.
2. Recommend whether one or two treatment plants should be built.

3. Determine the necessary size and location of connecting or intercepting sewers.

4. Recommend a suitable site or sites.

5. Prepare a reliable estimate of construction and operating costs.

At present there are two county-owned and operated sewage screening and chlorination plants located in the city of Yonkers. These facilities, located on the easterly shore of the Hudson River, serve the tributary areas. The South Yonkers sewage treatment plant is at the south line of Westchester County and the North Yonkers facility is located on the Hudson River about 1.5 mi. above South Yonkers. The southerly plant serves an area of 56 sq. mi. and the northerly plant 32.5 sq. mi. Design flows were respectively 84 and 50 m.g.d.

The consulting engineers, after studying in detail all the previously mentioned items, submitted a report on July 30, 1954. This report recommended:

1. A sewage treatment plant to accomplish the required degree of treatment, should consist of aerated grit chambers, aeration tanks with a detention time of 30 min., sedimentation tanks with about 2 hr. detention, chlorination facilities including a contact tank having 20 min. detention and an outfall into deep water in the Hudson River. Sludge thickening, digestion and storage tanks, and a barge of about 1,600 tons capacity for disposal of digested sludge solids at sea should be provided.

2. This sewage treatment plant should be constructed on a site consisting of land reclaimed by filling operations in the Hudson River opposite the existing South Yonkers sewage treatment plant. Sewage would be received by gravity through a 78-in. connecting sewer from the southerly sewage treatment plant and through a 54-in. force main from the existing North Yonkers treatment plant, which would be converted to a pumping station.

3. The estimated construction cost of the project outlined, based on an *Engineering News-Record* Cost Index of 650, was $10,500,000. Should the Cost Index rise to 750, the project cost would become $12,100,000.

In November, 1954, the electorate of Westchester County voted to authorize a $15,000,000 bond issue to cover the cost of the project. Consulting engineers were retained to prepare construction drawings for the project, except for some items to be designed by the county engineering forces.

At present, the acquisition of land is under way, one section of the connecting sewer design is about ready for bids, and it is planned to start construction work on site preparation early in 1956. The New York State Department of Health are permitting postponement of the construction of the pre-aeration features of the plant. Space is to be reserved, however, for this construction at a later date, if operating results demonstrate the need. An interesting feature of design is the use of sludge thickening and high rate sludge digestion. It is estimated that for a 60 per cent removal of solids in 1980 the digester tank loading will be 8.8 and 11.0 lb. of solids per cubic foot per month for the annual average and maximum month quantity of solids added, respectively. This is equivalent to about 0.3 cu. ft. per capita for the average loading.

DISCUSSION

By James M. Brown

In this paper the design features and initial operation problems of the sewage treatment plant for the New Rochelle Sanitary Sewer District will be described.

Design Flows

Although fairly tight intercepting sewers could be built, they must serve existing municipal sewerage systems, portions of which were built many years ago. Consequently, considerable infiltration and some storm flow could be expected. Intercepting sewers and the sewage treatment plant outfall were designed for the year 2000 and the treatment plant for the year 1975.

Population and sewage flow studies indicated that provision should be made for interceptor flows as shown in Table I.

The total population tributary to the new plant in 1975 would be 81,000. Wet weather flow in that year is expected to be at the rate of 32.2 m.g.d. and 44.8 m.g.d. in the sewer design year of 2000. The wet weather flow in 1975, was considered to average nearly 400 gal. per capita per day.

It was decided to provide maximum pumping capacity at the plant for 50 m.g.d. with provisions for discharging any flow in excess of 31.5 m.g.d. directly to the outfall sewer. The 31.5 m.g.d. flow could be pumped to the sedimentation tanks at a higher head if desired.

TABLE I.—Population and Sewage Flows, Westchester County, N. Y.

Flow Point	Population 1975	Flow (m.g.d.)	
		Wet Weather 1975	Sewer Design 2000
Larchmont Pumping Station	6,735	2.5	3.12
Easterly Interceptor	11,000	4.40	5.62
Westerly Interceptor	37,695	15.23	20.54
Stephenson Run	32,303	12.57	18.64
Plant total	80,998	32.20	44.80

Treatment Units

A primary type sewage treatment plant was designed and built to serve a population of 81,000 by 1975 at an average dry weather flow of 15 m.g.d.

Four vertical centrifugal pumps pump the raw wastes to the grit chamber where it flows by gravity to the comminutors and thence to the rectangular settling tanks. The primary tank effluent is chlorinated prior to discharge through a 54-in. concrete outfall line extending 9,900 ft. into Long Island Sound at a depth of 40 ft.

Pumping Facilities

Wet well design was predicated on the use of two constant speed pumps and two variable speed pumps with magnetic drives (Figure 1). Sewage in the wet well is held at a constant level by increasing or decreasing the intensity of the magnetic flux and hence the capacity of the variable speed pumps. The use of these pumps permitted designing the wet well for a 1 min. detention period at the maximum flow, rather than the customary 5 to 15 min. The resulting even flow through the treatment plant will provide greater settling efficiencies as well as more economical power usage. Included in the raw sewage pumping discharge system are four 24-in. cone valves wired to open after the centrifugal pumps start and to close slowly prior to the actual shutting down of the pumps. This prevents the transmission of water hammer and excessive pressure to the piping system; particularly the long outfall sewer when flushing it by means of direct pumping with all four pumps. Each pump has a capacity of 10.5 m.g.d. when pumping to the sewage

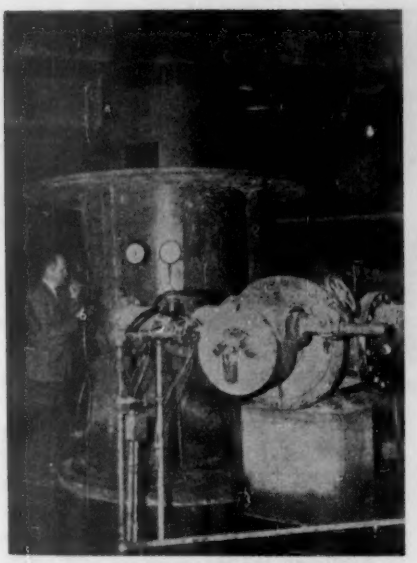

FIGURE 1.—One of four magnetic drive 10.5 m.g.d. raw sewage pumps.

treatment plant, providing a maximum wet weather flow of 31.5 m.g.d. When the four pumps discharge directly to the outfall sewer they have a combined capacity of 50 m.g.d. This capacity can be maintained for short periods of time by opening the drain valves from the sedimentation tanks to the wet well. This provision permits periodic flushing of the long outfall.

Primary Units

Grit removal is handled by a detritor with a wet weather detention period of 32 sec., and a design flow detention of 80 sec. Four rectangular settling tanks (Figure 2), each 156 ft. by 41 ft. have an average depth of 7 ft. and provide 2-hr. detention at the design flow. Effluent wiers of the V-notch type provide a designed overflow rate of 15,000 gal. per linear foot per day. The tank surface loading is 600 gal. per square foot per day. Linear flights, cross collectors and scum removal mechanisms remove the settled sludge to the sludge hopper. Sludge is removed to the digester from where it is pumped by two scrupeller pumps or by a piston pump. Grease and skimmings are automatically skimmed to the grease trough by rotating scrapers. The trough discharges to a

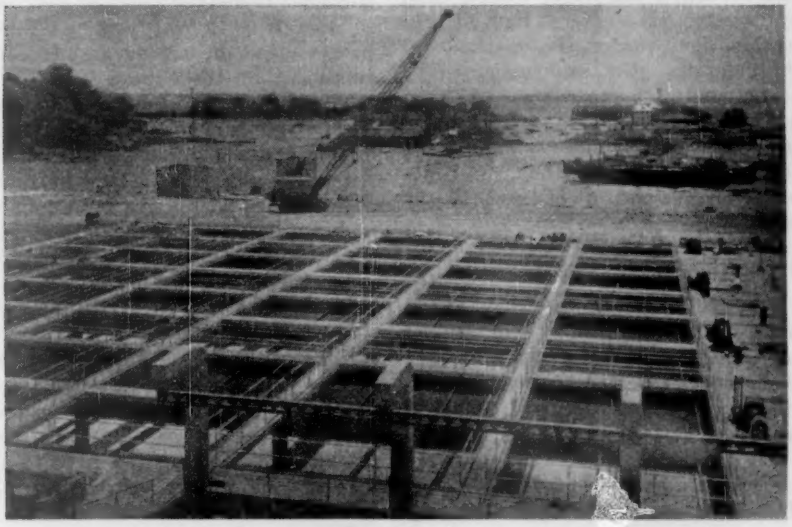

FIGURE 2.—Sedimentation tanks for 15 m.g.d. design flow.

scum pit from which skimmings are pumped to the digester by a heavy duty piston pump. A time clock schedules the operation of the skimmers and the piston pump is float controlled. Great flexibility has been worked into the design of the sludge pumping system. The operation of all sludge pumps as well as the hydraulic suction valves from each settling tank may be operated manually or by remote control from the digester penthouse. This enables the operator to control the pumping of sludge at the digester where he can visually inspect the sludge pumped.

Digesters

The four fixed covers 65-ft. diameter digesters each have a side water depth of 20.25 ft. and provide 3 cu. ft. of sludge capacity per capita. Flexibility of sludge piping permits operation of one or all of the digesters as primary tanks with the others being used as secondary units. The raw sludge or supernatant can be selected from any of three depths and heated in an external heat exchanger located in the basement of the digester building. Hot water from a hot water heater, fired by digester gas or No. 2 fuel oil in a combination burner, is re-circulated through the heat exchanger. The heated sludge or supernatant can then be returned through the overflow box at the top of the digester. Each digester is equipped with three draft tube mixers and a make-up water tank.

Other Units

Post chlorination for disinfection and pre-chlorination for odor control in proportion to the flow is provided by three chlorinators having a combined chlorine capacity of 6,000 lb. per day.

The treatment plant has two water systems in addition to a potable supply. A fresh water supply entirely separated from the potable supply is used for pump seals. A settled sewage supply is provided for cleaning purposes and for chlorinator injector operation.

Sludge will be dewatered on an 8 ft. by 10 ft. coil filter and the sludge cake will be flash dried and incinerated. Chemical conditioning with ferric chloride and lime has been provided.

No immediate attempt will be made to recover dried sludge for fertilizer. However, only minor changes to the present equipment will provide for the production of dried sludge if it is found advantageous. Tanks for elutriation of the sludge have been constructed, but the installation of the equipment has been postponed.

Initial Operation

Although the interceptors and treatment plant were not completed by the start of the summer bathing season, partial operation was commenced in May by pumping the flow from the New Rochelle Plant into the new and longer outfall. In August, the settling tanks and digesters were ready for operation and primary treatment was started. It has not been an easy task to operate with construction still in progress, but the urgency of additional treatment of the sewage during the bathing season and the relief of sewer overflows made partial operation desirable at the earliest opportunity.

Operation has for the most part been accomplished with a minimum number of headaches. Shortly after the main sewage pumps were started a considerable decrease was observed in the capacity of one pump. Examination showed that a piece had been broken off each edge of the nickel steel impeller. No foreign object was found within the pump casing and the exact cause of the damage has not been determined. During the first month of operation a drive sprocket on the cross collectors was found broken.

It was not feasible to obtain digested

FIGURE 3.—New Rochelle sewage treatment plant, Westchester County, N. Y.

sludge to seed the digesters. Two digesters, therefore, were placed in operation by pumping raw sewage into them through the heat exchanger. The sludge was added routinely five times per day.

Since the plant was still in the construction stage, cooperation had to be maintained with the unions on the job. This made it necesary to shut off the hot water boiler which provides heat to the digesters at 4:30 PM each day and to refire at 8:30 AM when the union returned to work. This has been ruled necessary since the filter and incinerator building which houses the boiler is still under construction and union men are present. Maintaining digester temperature above 80° F. under these conditions has not been possible.

Fortunately the full sewage load has not been turned into the plant and the digesters are operating at the low solids loading of approximately 0.9 lb. of volatile solids per cubic foot per month. No difficulty is anticipated with this loading at the lower digester temperatures. Volatile acids have remained below 1,350 p.p.m. and, although combustible gas has not been produced yet it is expected to start soon.

A well-trained lady chemist is responsible for an unusually large and well equipped laboratory.

Design of the intercepting sewers and sewage treatment plant (Figure 3) as well as the supervision of construction was under the immediate charge of L. G. Rice, Consulting Engineer to Westchester County. The Deputy Commissioner and Commissioner of Public Works have general supervision over the work.

RESEARCH ON ACTIVATED SLUDGE

VII. Efficiency of the Diffused Air System

By A. Pasveer

Research Institute for Public Health Engineering, National Health Research Council, The Hague, Netherlands

In this paper, the results obtained from the research on biochemical oxidation (1) and oxygenation of water with air bubbles (2) are applied to the activated sludge process of sewage purification.

The cost of the aeration process in the purification of sewage is mainly determined by three factors:

1. Serviceability of the oxygenation process.
2. Maintenance cost of the treatment plant.
3. Construction and equipment costs of the aeration tanks.

This paper presents some observations on the preceding factors as they relate to sewage aeration by means of compressed air.

Process Serviceability

With the aid of the formula $Q = 2\ A\ k\ \chi\ c_{L_s}\ t$ can be calculated V_{dh}, that is, the percentage of oxygen that is dissolved from an air bubble after passage through a water column h (the water having an oxygen content equal to zero). If the amount of energy that is required to introduce the air into the water column is also calculated, then the data are available from which the yield in grams of oxygen per kilowatt hour can be deduced.

The energy required is calculated using the formula:

$$W = 73{,}600\ \frac{Q\ (h + a)}{60 \times 75 \times r} \qquad (1)$$

in which

W = the expended energy (watts);
Q = the amount of air displaced (cu. m. per minute);
h = the height of the water column (mm.);
a = the overpressure * (mm. head of water);
r = the efficiency of motor and blower (per cent).

As a result, the yield in grams of oxygen per kilowatt hour is:

$$\frac{9769 \times V_{dh} \times r}{(a + h)}$$

in which V_{dh} and r are in percentages; and $(a + h)$ in mm. of water column.

The calculation of the yield in grams of oxygen per kilowatt hour was determined for air bubbles of from 0.2 to 1 cm. The results are summarized in Table I.

The following factors determine the results of this calculation:

1. The factor χ is the relative concentration gradient at the air bubble/water boundary at the moment that the contact has just ceased to exist. The value of χ was assumed for all air bubbles to be 1,300 per cm. For the largest air bubbles this value is probably too high, so that the results calculated for these air bubbles are better than in actual practice. Up to the present there have not been sufficient

* Resistance in the air diffuser and conduits.

TABLE I.—Effect of Different Size Air Bubbles on Oxygenation (Output)

Long Axis of Air Bubble (cm.)	$\frac{A}{V_b}$	h (cm.)	V_{da} (%)	Output (g. oxygen per kw.-hr.)	Long Axis of Air Bubble (cm.)	$\frac{A}{V_b}$	h (cm.)	V_{da} (%)	Output (g. oxygen per kw.-hr.)
0.20	31.8	50	10.4	16,933	0.50	15.5	50	5.2	8,466
		100	19.7	17,495			100	10.1	9,058
		150	28.1	17,157			150	14.8	9,036
		200	35.5	16,514			200	19.3	8,978
		250	42.3	15,893			250	23.5	8,830
		300	48.3	15,220			300	27.5	8,666
0.25	26.5	50	8.7	14,165	0.60	13.8	50	4.7	7,652
		100	16.7	14,831			100	9.1	8,082
		150	24	14,653			150	13.3	8,120
		200	30.7	14,281			200	17.4	8,094
		250	36.7	13,789			250	21.2	7,965
		300	42.3	13,330			300	24.9	7,847
0.30	23.4	50	7.7	12,537	0.80	11.5	50	3.9	6,350
		100	14.9	13,232			100	7.6	6,749
		150	21.5	13,127			150	11.2	6,838
		200	27.6	12,839			200	14.7	6,838
		250	33.2	12,474			250	18	6,763
		300	38.4	12,101			300	21.2	6,681
0.40	18.8	50	6.3	10,257	1	10	50	3.4	5,536
		100	12.2	10,835			100	6.7	5,950
		150	17.7	10,807			150	9.8	5,983
		200	22.9	10,653			200	12.9	6,001
		250	27.7	10,408			250	15.9	5,936
		300	32.3	10,179			300	18.7	5,893

Notes:

1. It is assumed that (has the same value for all sizes of air bubble.
2. The efficiency of motor and blower is assumed to be 100 per cent.
3. The output in grams of oxygen per kilowatt hour is computed for a pressure loss in the air diffuser equal to a 10 cm. water column.
4. The output in grams of oxygen per kilowatt hour is computed for a zero oxygen content in the water.
5. The output is computed for an air bubble ascending in quiescent water.

observations of (for these bubbles. (For bubbles of 1 cm. of value (= 1,018 per cm. was found).

2. The V_{da} was calculated for an oxygen content of zero, a temperature of 10° C. and a barometric pressure of 760 mm. Thus the yield gives the number of grams of oxygen per kilowatt hour. The yield for an oxygen content of the water = c mg. per liter is obtained by multiplying by the factor $(11.25 - c)/11.25$. No correction is required for differences in temperature, as the influence of a lesser deficit at a higher temperature is compensated for by the higher value for the diffusion coefficient.

3. The efficiency of the motor and blower was assumed to be 100 per cent. In reality this efficiency is between 60 and 80 per cent.

4. The resistance (a) in the air diffuser and conduits has been considered to be equal to a water column 10 cm. high. It is not known whether or not this value can be applied in practice for small air bubbles 0.2 and 0.25 cm. in diameter. For the larger air bubbles this slight overpressure applies only in the case of new air diffusers. The resistance of the air diffuser increases during use and may often equal several meters of water column. The actual yield of oxygen is then much less than that shown in Table I.

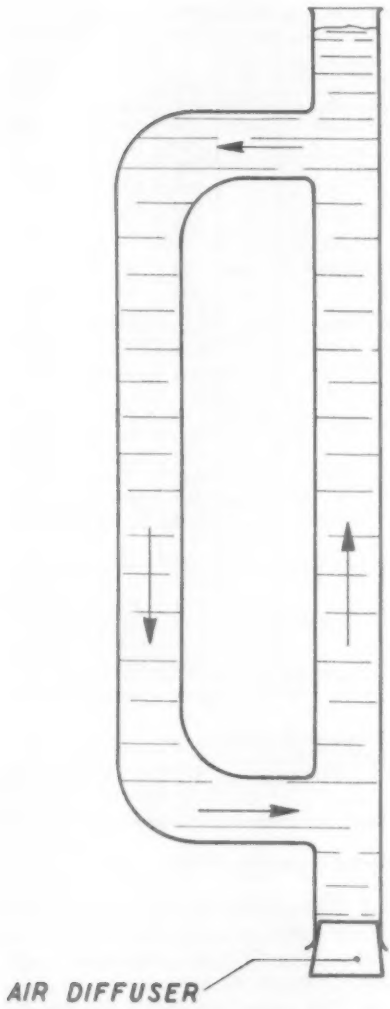

FIGURE 1.—Sketch of glass tube used in laboratory studies on the effect of water circulation on oxygenation.

5. The V_{dA} (and hence the yield) was calculated for the ascent of air bubbles in quiescent water. In the diffused air activated sludge process, the spiral flow system in particular and maybe to a somewhat lesser extent in the ridge and furrow system, the circulation in the aeration tank is such that the air bubbles ascend in water having quite a high upward velocity. As a result the time of contact of the air bubbles with the water is shorter and therefore the amount of oxygen dissolved is less.

This condition was verified in the laboratory by comparing the yield in the tube shown in Figure 1 with the yield in a straight tube of the same height. In the tube in which the water is kept circulating, the yield is as much as 30 to 35 per cent less than in the straight tube. In the activated sludge tank the vertical velocity of the water is probably even greater.

It is found by calculation that approximately 9 per cent of the oxygen in the air bubbles will have been absorbed in the water:

$$\frac{2}{3} \times \frac{8.25}{11.25} \times 18.7 = 9^{+} \text{ per cent}$$

For this calculation it is assumed that the head of water is 3 m. and the mean oxygen content in the aeration tank is 3 mg. per liter. The average size for the air bubbles was 1 cm. and a decrease in yield of one-third was included as a result of the shorter air water contact time due to circulation of the tank contents.

This percentage agrees closely with the average value of 10 per cent given by Imhoff (4). The same value of 9 per cent may also be determined by taking a smaller average diameter for the bubbles, with a greater decrease in yield due to circulation. This will be more in accordance with actual conditions as follows from the work of King (6) who estimated high tank circulation velocities.

6. There is the possibility that in sewage a film of substances possessing surface activity is formed around the ascending air bubbles; as a result the transfer of oxygen is less. This effect has not been considered in the preceding calculation.

It is evident from the previous statements that although the theoretical

yields from the process of aeration with compressed air are high, various conditions must be taken into account which make the ultimate yield much lower. When the resistance in the air diffuser is high, the yield in compressed air aeration is not higher than with the rotating brush. (Oxygenation capacity equal to approximately 1,500 to 2,000 grams of oxygen per kilowatt hour.)

This is in agreement with the observation that in an activated sludge plant with brush aeration the consumption of energy is usually somewhat less than in a compressed air installation (5).

It is important to investigate whether or not the yield might be improved and in what way. There are three points deserving particular attention, *viz.*, the air diffuser, the depth of the aeration tank and the circulation.

Improving the Yield

Air Diffuser

Table I shows how essential it is for the air to be divided as finely as possible. Air dispersion is limited in varying degrees by the clogging of the air diffusers. Clogging is more marked the greater the fineness of the diffuser pores. Aerating with clean air aids considerably in preventing clogging. It would be desirable to continue research until a method is found whereby finely divided air bubbles can be produced at a low consumption of energy and in which no risk of clogging occurs.

Aeration Tank Depth

A number of factors must be considered in selecting the depth of the aeration tank, in addition to the cost of energy. These factors require examination on an individual basis.

Table I shows that from the point of view of energy cost, especially in the case of smallest air bubbles, the depth at which air is introduced into the water should be small (1 to 2 meters) for low air diffuser resistances. At first this would seem to be contrary to the view that at a greater depth, owing to the high pressure of the air bubble, the speed of solution of the oxygen must be greater. This is actually correct, but the more rapid dissolving of the oxygen is balanced by the fact that at a greater depth the number of bubbles that can be formed from a given volume of atmospheric air is proportionally less. Of course, the total oxygen absorption in the case of the more shallow tank is less. An increase in the surface area of the aeration tank or the use of more filter plates in the tank bottom area is required.

Tank Circulation

Aeration tank efficiency can be increased by reducing the upward velocity, and in turn the circulation. This can be achieved by distributing the air supply uniformly over the entire area of the tank bottom.

Operating Cost Considerations

Diffuser Clogging

The weak point in the diffused air system with porous air diffusers is that usually considerable difficulty is caused by clogging of the air diffusers. There are extensive references to this subject in the literature.

Many attempts have been made to replace the air diffusers with tubes having holes or slits. It has been reported that in many cases in activated sludge tanks these holes also become clogged (5). This was experienced by the author in a pilot plant in which aeration was effected through holes having a diameter of 2 mm. and spaced 6,000 per square meter.

Large Bubble Aeration

In the course of further research the expedient of using much larger apertures has been tried. The air was introduced into the water through

apertures 3 cm. in diameter. This method of aeration offers the great advantage that the treatment plant can be a simple type and there is no difficulty with clogging of the apertures. Furthermore, the cost of maintenance is low. A great disadvantage of this method is its low efficiency as the yield in grams of oxygen per kilowatt hour is small. This is due to the small absorption of oxygen from the large air bubbles. Also the rapid circulation of the tank contents results in a very short contact time between the air bubbles and the water.

Some investigators are of the opinion that with this method of aeration most of the oxygen is absorbed at the surface of the tank. Pöpel (7), for instance, takes this for granted in the designing of aeration tanks.

It is important to obtain some idea of the extent to which oxygen is absorbed at the tank surface. This is necessary in order to assess the value of large bubble aeration and to ascertain whether or not short contact times are wholly or partly compensated for by rapid oxygenation at the tank surface in the case of aeration tanks with rapid tank circulation. Previous observations of a tank with a horizontal stirring device have already revealed that the surface aeration contribution is quite small even with rapid tank circulation. It seemed desirable, however, to determine the value of surface aeration for a tank using the large bubble aeration method.

The oxygenation capacity was determined at different rates of air supply in the concrete tank (Figure 2). The observed increase in oxygen and the calculated oxygenation capacities are summarized in Table II. Also shown is the trend of the oxygen content when nitrogen instead of air was passed through the water. In this case precautions were taken so that the rate of supply of nitrogen was as nearly as possible the same as in the corresponding experiment with air. If the absorption of oxygen at the tank surface is considerable it should be revealed in these experiments with nitrogen.

The results show that when nitrogen is passed through water under the conditions selected, the oxygen content of the water (about 1.5 mg. per liter) in the aeration tank undergoes little change. It follows that in these experiments there was a virtual equilibrium between the rate of absorption of oxygen at the tank surface

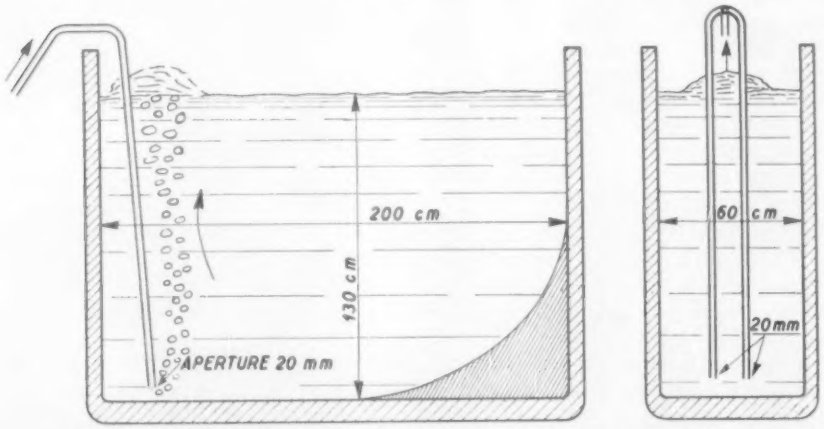

FIGURE 2.—Sketch of aeration tank used to determine oxygenation capacity at different rates of air supply.

TABLE II.—Experiments With Air and Nitrogen for Determining the Effect of Aeration Tank Surface on Oxygenation

Item	Air Supply (cu. m. per hr.)						Nitrogen (cu. m. per hr.)					
	11		7.5		4		13		9		4	
Height of water column (cm.)	130		130		130		130		130		130	
Tank volume (cu. m.)	1.5		1.5		1.5		1.5		1.5		1.5	
Temperature (°C.)	11.0		11.1		11.8		11.8		11.1		11.8	
Time (t, min.)	t	mg. O_2/l.	t	mg. O_2/l.	t	mg. O_2/l.	t	mg. O_2/l.	t	mg. O_2/l.	t	mg. O_2/l.
	0	0.40	0	0.58	0	1.73	0	1.55	0	1.67	0	1.54
	2	1.88	2	1.39	3	2.85	2	1.48	2	1.69	3	1.57
	4	3.47	4	2.60	6	3.30	4½	1.35	4	1.62	6	1.71
	6	4.68	6	3.35	9	4.10	6	1.37	6	1.56	9	1.61
	8	5.77	8	4.29	12	4.70	8	1.30	8	1.68	12	1.63
					15	5.11					15	1.70
tan α	1.88		1.16		0.72							
O.C. (g. per cu. m. per hr.)	47.8		29.4		18.1							
g. oxygen per net kw.-hr.	appr. 1,300		appr. 1,000		appr. 1,500							

and the rate at which the oxygen is removed from the water by the nitrogen bubbles.

Stefan's formula

$$Q = 2\,A\,(c_{L_s} - c_L)\sqrt{\frac{kt}{\pi}}$$

for the solution of oxygen in water also holds good for the removal of oxygen from water. In this formula, for an atmosphere of nitrogen, c_{L_s} is replaced by the oxygen concentration in the water mass (c_L) and c_L by the oxygen concentration in the boundary surface, which in the atmosphere of nitrogen may be taken as being practically equal to zero.

Provided steps were taken to insure an equal speed of passage of air and nitrogen, it may be assumed that the resulting boundary surface (gaseous atmosphere-water) was the same in both cases. The equation for the oxygenation capacity (O.C.) deduced with the aid of Stefan's formula is

$$\text{O.C.} = 25.9 \tan\cdot\alpha \sqrt{\frac{k_{10}}{k_t}} = 2.303 \times 11.25 \tan\cdot\alpha \sqrt{\frac{k_{10}}{k_t}}$$

and therefore, may be used also for the speed of oxygen removal from the water together with the nitrogen, if the value 11.25 (c_{L_s} at 10° C. and 760 mm. barometric pressure) is replaced by the oxygen content of the water. In the equation for the oxygenation capacity $\tan\cdot\alpha$ is a variable governed by an expression which is a measure for the area of the air—water surface and the degree of renewal of this boundary surface (3).

Based on these theoretical considerations an experiment was conducted passing nitrogen through an aeration tank filled with water having a very low oxygen content. The nitrogen supply rate was 7.5 cu. m. per hour and maintained for a period of 40 min. The oxygen content of the water in the tank had then reached the equilibrium value of 1.10 mg. of oxygen per liter at 12.9° C. By using the value $\tan\cdot\alpha = 1.16$, (Table II) the speed of removal of oxygen by nitrogen at 10° C. can be calculated by the expression:

$$-\frac{dc}{dt} = 2.303 \times 1.10 \times 1.16 \times 0.948 = 2.79 \text{ g. of oxygen per cubic meter of water per hour}$$

in which

$$0.948 = \sqrt{\frac{k_{10}}{k_t}}$$

The speed of deoxygenation is equal to the speed of oxygenation at the

tank surface at 10° C. The oxygenation at the tank surface (2.79 g. per cubic meter per hour) does not amount to as much as 10 per cent of the 29.4 g. per cubic meter per hour which is supplied when air is blown through the water (Table II). Moreover, it must be remembered that in practice the height of the water column is at least twice the height used in these experiments. Accordingly the surface aeration contribution expressed as a percentage is reduced to one-half.

Thus it is found that the absorption of oxygen at the tank surface, even with large bubble aeration in which the surface turbulence is more violent than in the case of aeration with small bubbles, is unimportant. Even at low speeds of oxygenation in which the percentage of oxygen contributed by the tank surface may be a maximum, it will not often exceed 10 per cent.

The low efficiency of large bubble aeration therefore is not compensated for by a substantial absorption of oxygen at the aeration tank surface.

The question now to be answered is: can large bubble aeration be improved in such a way that its efficiency does not show too great a disparity with other methods of aeration? If this can be done the advantages of simplicity and low maintenance costs would be fully realized.

Location of Diffusers

It also follows that in this case increased efficiency will be effected mainly by removing the cause of the short contact time, *i.e.* by eliminating the circulation. An experiment for orientation in this direction is described.

Air was supplied to the tank (Figure 2) through four openings. In the first series of tests these openings were placed along the short side of the tank, in the second series of tests the tank bottom area was limited to 111 cm. by 60 cm. and the four openings were distributed over this bottom area.

In Table III the results of these experiments have been combined. With the lower air rate and less turbulence, there is no difference in the two methods of air supply. There probably was circulation in both cases.

At the higher rates of air supply, however, there was an increase in oxygen output per kilowatt hour up to 13 per cent. It might be that with still more openings per unit area of tank bottom and with deeper tanks the difference between both kinds of air supply may be more.

Construction and Equipment Costs

The construction and equipment costs must be considered in the annual cost of the treatment plant.

In previous research (3) it was found that with rotating brush aeration, complete purification of a 6 to 8 times normal amount of sewage could be obtained by merely increasing the rate of oxygen supply to 6 to 8 times the usual value.

In subsequent laboratory experiments (1) it was shown that even an in-

TABLE III.—Experiments with Large Bubble Aeration

	Aeration From Four Openings at One Side of Tank			Aeration From Four Openings Distributed Over Tank Bottom		
Head of water (cm.)	130	130	130	130	130	130
Volume of water (cu. m.)	1.5	1.5	1.5	0.88	0.88	0.88
Volume of air per opening (l.)	0.62	0.86	1.15	0.62	0.87	1.17
O.C.[1] (g. per cu. m. per hr.)	32.5	51.3	68.5	55.3	97.6	134.7
Output (g. oxygen per net kw.-hr.)	1,465	1,622	1,630	1,430	1,797	1,843

[1] Oxygenation capacity.

crease of 50 to 100 times the normal amount of sewage can be completely purified provided the oxygenation capacity to load value remains constant.

On this basis an increase of 10 to 20 times in rate of oxygenation is not an extreme measure. The load per unit volume thereby may be increased 10 to 20 times. The requisite aeration tank volume is accordingly reduced to 5 to 10 per cent of what is considered necessary at present.

It would for instance be possible, in a tank with diffused air aeration, to provide the whole tank bottom with porous air diffusers. This may be accomplished by providing 10 to 20 times as many porous air diffusers per square meter of tank bottom. With the same speed of air passage the speed of oxygenation would be increased 10 to 20 times. The increase in oxygenation would in fact be more than this as a result of the improved efficiency due to the absence of circulation.

When existing purification plants are being enlarged it will in principle be unnecessary to increase the volume of the aeration tank. Increasing the intensity of oxygenation will make it possible to increase the load per unit volume of the aeration tank.

This method must, of course, be tried under practical conditions. Efforts will be made to carry out an experiment on a practical scale with an aeration tank equipped in the manner previously indicated.

Conclusions

1. Aeration with small air bubbles is more efficient than with large bubbles. The output in grams of oxygen per kilowatt hour in the case of small air bubbles may be considerably higher than in the case of larger air bubbles.

2. The rapid tank circulation in an activated sludge tank with a diffused air system aeration has a decidedly detrimental effect on the amount of oxygen passing into solution.

3. The absorption of oxygen at the surface of the aeration tank proved to be of minor importance. This also applies when aerating with large air bubbles. In the case of low oxygenation capacities the effect of surface aeration is high, but even then, in tanks with a depth of 2.5 to 3 m., it will not be more than 10 per cent.

4. In the case of low air diffuser resistance, shallow aeration tanks (1.5 to 2 m. in depth) are more economical than deeper tanks, especially when aerating with small air bubbles.

5. Special attention must be given to the possibility of decreasing the aeration tank volume by increasing the rate of oxygen supplied to the manifold. The detrimental effect of rapid circulation can then be avoided.

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IRRIGATION WITH SEWAGE EFFLUENTS

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The land treatment of sewage is as old as sewage disposal itself. Until recently, however, sewage farms were neither well planned nor skillfully managed. The disposal of sewage effluent was the primary object; the raising of crops and replenishing of ground water generally being considered of secondary importance. As economy of installation and operation usually resulted in too much sewage being used on too little land, septic conditions were frequently produced. It is not surprising that the old practices of land disposal of sewage have been largely abandoned.

The new methods of using sewage effluents for planned irrigation, with emphasis placed on conservation and on agricultural utilization, is a recent development. This practice returns to the land as much as possible of the organic matter and fertilizing elements that have been withdrawn from the soil in the harvesting of foodstuff, and results in superior growth of field crops. The new technique utilizes a low rate of application over large areas of land, integrated with the planning of agricultural crops, and requires proper maintenance and operation of all irrigation and agricultural devices.

While new methods have proved workable in the last two decades, there are still many problems worthy of investigation. It may be possible, for example, to increase the irrigation rates in some climates and on certain types of soil (governed only by the plant demand for moisture). For the treatment of sewage from small communities, it may be possible to combine planned irrigation with a new type of septic tank. The technique employed at Cologne, Germany, indicates the utility of a two-compartment septic tank (1). By providing short fermentation and aeration periods, this type of septic tank stabilizes even slightly putrified sewage; the effluent is suitable for irrigation in close proximity to human habitations, without creating a sanitary hazard or a nuisance.

Experience at Leipzig

The city of Leipzig has been faced with an unusually difficult sewage treatment problem, as the total flow of streams in the region was approximately equal only to the volume of sewage effluent being discharged into the rivers. For this reason, many properties formerly were not connected to the municipal sewerage system. In 1935, as a result of a special governmental act, which enables the financing of agricultural utilization of sewage, the City Council decided to carry out a planned program for land treatment of sewage and industrial wastes. To accomplish this, the city of Leipzig made an agreement with a cooperative of farmers (200 responsible shareholders involving about 1,600 members) for handling the sewage throughout the year. The average flow at that time was 26 m.g.d. Two years after the original agreement, another cooperative was established; this involved the disposal of an additional 3 m.g.d. of sewage by land irrigation.

According to its agreement, the city is to supply sewage effluent, after providing primary treatment, to the highest level of the irrigation plateau. Two pumps having a total capacity of

FIGURE 1.—Carrier ditch has concrete lining laid with asphalt joints.

9,156 cu. yd. per hour discharge the sewage into the effluent storage tank through a pressure pipe, 3.6 ft. in diameter and 8.4 miles in length, to an elevation of 125 ft. From this point, it is the responsibility of the cooperative to distribute the effluent over the irrigated land. The system of ditches used totals approximately 220 miles in length (see Figures 1 and 2), distributing effluent to 57,000 acres of irrigated land.

Five main brooks divide the irrigation plateau into four irrigation districts. Each district is supplied by one principal carrier ditch, 6.5 ft. in width and ranging in length up to 40 miles. For crossing brooks, roads, valleys and railroad lines, the effluent is conducted through pipes of various diameters (from 8-in. to 4.0-ft.) To distribute the effluent over farms for surface irrigation of grassland, and for ridge-and-furrow irrigation of vegetables, 6,000 outlets were planned (see Figure 3). In recent years (2), more and more use has been made of spray irrigation methods in which mobile pumping and spraying units are employed.

FIGURE 2.—Effluent flow regulation and discharge structure in irrigation system.

Pretreatment Practices

Some form of sewage treatment, as by plain sedimentation (30 min. detention) to remove coarse solids, usually precedes irrigation. To deliver the maximum organic matter to fields, comminutors must be employed. Experience has shown that complete primary treatment may be necessary to precede agricultural use of sewage effluent. The advantages are: less wear on pumps; reduced sludge deposits in ditches and pressure sewers; prevention of hydrogen sulfite production (poisonous to plants); no clogging of soil by undigested sludge solids; and opportunity to rest drainage fields if the alternate discharge of effluent to a stream is feasible.

FIGURE 3.—Distribution outlets provided for surface irrigation of grassland.

According to data published by Reinhold (3), the fertilizing value of an average sewage effluent in Berlin was approximately the same for primary treated as for raw sewage. Sewage effluent following secondary treatment had lost a substantial part of nutrients, as shown in Table I.

Secondary treatment reduces the fertilizing value of the nitrogen content of effluents because decomposition and removal of suspended nitrogeneous compounds takes place.

TABLE I.—Nutrient Values of Sewage

Sewage	Total Nitrogen mg./l.	Total K_2O mg./l.	Total P_2O_5 mg./l.
Raw, untreated	66.2	42.1	22.6
Primary treated	73.3	43.2	23.5
Secondary treated	17.2	37.0	17.6
Drainage water from farm fields	12.7	20.0	8.4

Small communities located near inexpensive land sites may find it economically sound to use a simple type of open earth sludge digestion basin. Basins of this type have been in use in Germany (Wupper-and-Weisselster-Verband) for 15 yr. without interruption of service. The dimensions of these basins are based on providing a capacity of 4 to 5 cu. ft. per capita of contributing population. The digested sludge, removed twice each year from these basins, can be added to the effluent used for irrigation. There is no danger of clogging of soil, because the organic matter has been converted into humus; the nutrients in this form are immediately available as plant food.

Methods of Application

Where sewage effluent is pumped, it is necessary to provide storage basins if pumping is to be confined to the working hours of the day. Storage basins are included for that purpose in several recent installations designed to apply effluents to the land;

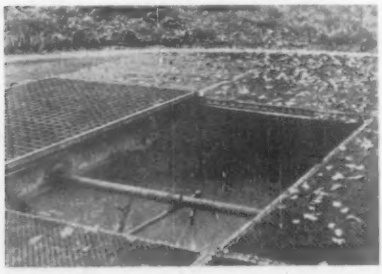

FIGURE 4.—Dairy waste storage tank.

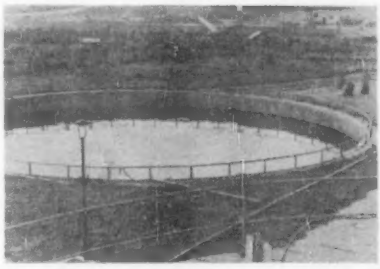

FIGURE 5.—Municipal sewage storage tank.

these installations handle both industrial wastes (Figure 4) and municipal sewage (Figure 5).

Many installations for the land treatment of sewage effluents and industrial wastes have been established in Western Europe. For example, since 1950 the small state of Bavaria has erected 28 new installations, and spent a million dollars in that year for the support of special pilot plants and research facilities. Recent legislation provides that a secondary sewage treatment plant can receive a state subsidy only after investigation has shown that land treatment is not feasible from the technical or economical point of view.

Especially good results are reported from the agricultural use of effluent for field irrigation at Coesfeld/Westfalen, a town of 15,000 population having a large textile industry. The textile processing wastes total 1.2 m.g.d. and constitute about 45 per cent of the total sewage flow. The sewage flows through a trash rack, and is pumped into a storage tank. The plots of land (approx. 240 acres) receiving the mixed wastes are used, almost exclusively, as meadows and pastures (4).

Irrigation methods should be adapted to local farming practices, nature of soil, climatic conditions, height of ground-water table, topography, and the type of crops raised. There are two principal types of sewage irrigation, the spray system and

the broad surface method; in addition there are the ridge and furrow system, the flooding method and the subsurface method.

Spray Irrigation

The most successful method of using effluents for irrigation is the spray system, because the effluent is distributed uniformly over the land being irrigated. Wind sometimes disturbs the uniformity of coverage, and laterals should not be set up so close to highways or adjacent property that spray can be blown onto them.

This new technique of sewage utilization has been developed in Germany and in the United States largely through the collaboration of equipment manufacturers and consulting engineers. Large revolving spray nozzles, attached to movable pipe lines, distribute sewage over the land. By this technique, crops are wetted as by rain; frequent application of small quantities of effluent is used, rather than in frequent heavy flooding. There are no losses of effluent through percolation in ditches, because the distributing system mostly is comprised of underground piping. Hydrants are established for the connection of movable pipes, or sometimes portable pumps are used for local spraying of effluent drawn from ditches (Figures 6 and 7).

The cover crop on the disposal area is a very important part of the spray irrigation system. It has been found that the best absorption of effluent,

FIGURE 6.—Spray irrigation by connecting movable pipes.

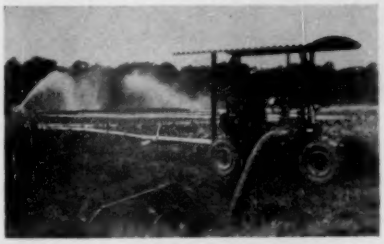

FIGURE 7.—Spray irrigation by pumping from ditches.

and the least runoff, is obtained from well established pastures or from fields covered with dense grasses or clover. The choice of cover crop to be used depends on geographical location, type of soil, and the use to which the irrigation land is put. In Europe favorable results are obtained, especially on sandy soils, with red clover, Luzerne grass and German pasture grass.

Spraying into grazing areas is hygienically and economically the best means of sewage utilization. Undisturbed natural growth of plants, such as grasses, also promote the development of good soil structure with optimum utilization and percolation rates. The spray irrigation system has been in continuous service in Western Europe for more than two decades, and has been entirely satisfactory from every viewpoint. Crop yields have been increased markedly through well-managed irrigation.

In the United States, the use of effluents for spray irrigation of crops is rather recent. Spray irrigation of milk processing waste was placed in operation in several states during the last five years. In addition, spray irrigation is used as a method of disposal for canning and frozen food wastes—industries which are seasonal in operation. Where conditions are suitable, spray irrigation may be the most economical and least troublesome method for the disposal of many organic industrial wastes.

A recently introduced method utilizing plastic pipe, together with

plastic tees and plugs, reduces costs of spray irrigation over other types of installation. The flexibility of the plastic pipe allows lateral lines to be moved easily from one location to another.

The favorable effect of dairy wastes applied to crops by spray irrigation can even exceed what would be expected from corresponding amounts of manure or commercial fertilizer. This may be due to the stimulating effect of polyuronic acids (formed from the carbohydrates in the waste) on favorable soil bacteria (5).

It has been found that 40 in. of effluent can be applied to a crop of Reed canary grass during the growing season. By this application, the plant nutrients in the effluent increased the crop yield by about one ton per acre on a mineral silt loan, and about two tons on peat. Drainage water from the irrigated soil did not increase the coliform index of the nearby creek, but did increase its chloride and sodium content (6).

At Seabrook Farms (7) in New Jersey, 3,500 acres are now watered by portable-type irrigation using vegetable processing effluent. This practice solves an industrial waste disposal problem, providing a source of water for crops and replenishing the ground water supply of the processing plant.

Broad Surface Irrigation

For broad surface or trench irrigation, the main carriers usually are concrete lined ditches. The minor distributing systems require carefully laid out earth channels, with concrete lining only where soil is especially permeable. It is desirable to distribute the effluent uniformly over the land being irrigated. If slopes are adequate (2 per cent), the effluent is flowed over the land from ridge distributors in the form of ditches or pipes having side outlets. Where slopes are flat, the sewage is carried from the distributors in furrows. Cross ditches are provided to collect sewage that has not percolated into the ground or that is not held back by the crops. The sewage collected by cross ditches may be redistributed over successively lower areas.

FIGURE 8.—Ridge-and-furrow irrigation illustrating even distribution.

In order to avoid pools of sewage, and consequent irregular growth of plants, the grading of land surface and furrowing must be carried out very accurately. Irregular distribution of effluent may result in over-irrigation, and may destroy a crop in low parts of the land. Suitable irrigation rates have previously been published (8).

Ridge-and-furrow System

A ridge-and-furrow system is generally used for the irrigation of vegetable crops. As the name implies, the system involves a series of parallel ridges and furrows. In Europe, the ridges are usually 3 to 4 ft. apart and 8 to 10 in. high, with furrows 10 to 12 in. wide and 150 ft. in length. If the nature of the soil requires it, an underdrain is laid under the center of each ridge. Sewage flows along the furrows and seeps laterally through the ground. The crops, which grow on the ridges, are irrigated by the sewage in the adjacent furrows. In this method of irrigation, the sewage does not come in contact with crops. Each furrow should receive the same amount of sewage, and the grade should be such that as much sewage is absorbed by the soil at the upper end of the furrow as at the lower (Figure 8).

Flood Irrigation

Flood irrigation consists of level fields surrounded by banks or low dykes. The area is alternately flooded with effluent and is rested; this method assures reaeration of the soil. Underdrains are used to carry off the effluent and to prevent ground water from rising above a certain level. This method is at times combined with ridge-and-furrow irrigation; the area is broken up into a series of beds lying between furrows of approximately equal surface area. These beds are used for raising crops which are not wetted directly by sewage. Flood irrigation is generally employed only as an emergency device when the spray or surface systems are out of use; for example, at night and during winter periods.

Drainage

No satisfactory irrigation is possible without drainage, and surplus water should not be allowed to accumulate. Poorly drained soil may so concentrate salts that crops cannot be raised, or an abundance of water may cause excessive leaching of plant food material from the soil. The amount of water a soil may hold will vary not only with the size of soil particles, but also with the distance of the water-table below the surface, and the frequency of rain.

Land areas that do not naturally drain off excess water must be drained artificially. Good drainage improves the physical condition of the soil, causing the retention of a larger amount of film water which in dry seasons is needed by the growing crops. In order to maintain a sufficient oxygen supply and to reduce carbon dioxide accumulation, the pore space of the soil must be kept in contact with the atmosphere (Figure 9).

Subsurface Irrigation

Subsurface irrigation involves distribution of the sewage effluent by

FIGURE 9.—Irrigation drainage ditch with effluent supply conduit in foreground.

open joint tile pipe, laid below the soil surface. A new type of pipe was developed for this purpose by Stein (9) and was used in the irrigation fields of Delitsch. This pipe is designed to serve both for irrigation and for drainage. The specially manufactured pipe ("Bitu-Pipe") has a solid bottom and a porous top. It is produced in a manner similar to tile pipe, but is somewhat higher in cost than clay pipe. It is more resistant to soil deterioration, to acid and electrolytic corrosion, and even to frost damages.

This type of pipe makes it possible to distribute sewage effluent uniformly over the full length of the line. Theoretically the laterals should be set level, but practically they are graded with a small slope as the pipes also serve for drainage. The spacing of laterals varies from 3 to 5 ft. in sandy soils, to 8 to 10 ft. in silt soil. The pipes should be laid not more than 2 ft. below the surface, because the distributed sewage effluent must reach the plant root level. This system enables greater root penetration and expansion, and correspondingly increases the yield of crops.

Irrigation Planning

As sewerage systems and sewage treatment are closely related with the future development of communities, the design for a full scale use of sewage effluents for irrigation should include evaluation of agricultural considerations. It is not always satisfac-

tory to collect all the sewage at one centrally located primary treatment plant. This is especially true in the large cities. Frequently it is better to consider several smaller treatment plants with adjacent irrigation areas.

Septic conditions often are produced in long outfall sewers in which a steady or self-cleaning flow is not maintained. The use of sewage effluents by farmers can be suspended only where available receiving streams provide a large factor of dilution. At some locations lagoon storage can be provided to handle the effluent during freezing weather. It is desirable to have sufficient areas of land available for emergency flooding.

Competent operation and adequate attention to details are essential for the successful use of sewage effluents in irrigation. A properly designed and constructed installation will be found to exhibit lower maintenance and application costs. Education of farm operators in the proper use of effluents for irrigation will be required.

If sewage effluent is supplied by spray irrigation during the growth of agricultural and horticultural crops, application should be stopped three weeks before harvest. Similarly, irrigation should be stopped 10 days prior to the use of land for pasturage, or before the harvest of grass or cattle fodder.

Areas of land must be available for emergency flooding. The proper agricultural use of sewage throughout the year can be assured only in those installations where receiving streams are available which provide a large factor of dilution. At times lagoon storage can be provided to handle the effluent during freezing weather.

Conclusion

Sewage effluent is available in all localities and can be utilized in certain areas for irrigation purposes; in water-shortage areas it may be extremely valuable. The simple spreading of effluent is usually recognized as an unsound economic venture (10). Skillfully designed and properly managed sewage farming has been operated for decades in Europe, and for several years in some localities in the southern areas of the United States, without danger to personnel and without creating nuisance conditions. As a new science, irrigation with sewage effluent is still in the experimental stage and many problems need to be solved in the fields of bacteriology, biology, hygiene, soil structure and the physiology of plants. Special pilot and research installations should be planned to study irrigation with sewage effluents and industrial wastes. Universities and research institutions could solve the scientific problems and develop the most effective ways for irrigation and the recharge of ground water aquifers.

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CONFERENCE OF STATE AND INTERSTATE WATER POLLUTION CONTROL ADMINISTRATORS

An important feature of the Federation's Annual Meetings is the opportunity for committees—both official and unofficial—to schedule meetings for business and general discussion. Much work of interest to the profession is carried on at this time. Among such noteworthy meetings was the Conference of State and Interstate Water Pollution Control Administrators on October 11, 1955. The meeting was attended by more than 50 persons from 28 states, 5 interstate agencies and 7 other groups.

The expressed purpose of this conference was to clarify the opinions of the water pollution control agencies concerning S. 890 * and, if possible, to arrive at a mutually acceptable position concerning this bill before it comes up for action at the 1956 session of Congress. The Conference took the following action on the several sections of the bill.

Sections 1 through 4 inclusive (Declaration of National Policy) were acceptable.

Section 5 (Grants-in-aid) There was a divergence of opinion on the advisability of such financial aid for administrative purposes; there appeared to be general acceptance of grants for research purposes. By a 16 to 7 vote, the majority agreed that the principle of grants-in-aid should be left in the bill.

Section 6 (Water Pollution Control Advisory Board) It was recommended that the number of state and interstate representatives at least equal those of the federal agencies.

Section 7 (Enforcement Measures) Four amendments were approved.

1. Provide a 90-day cooling off period when the Surgeon General finds there is cause of water pollution and before there is action taken. Give interstate agencies a chance to rectify the situation.
2. An additional 90 days should be allowed for consummating the control.
3. That if a state or interstate agency has issued an order before the Surgeon General does for the control of water pollution, no action shall be taken by the Surgeon General to enforce his later order until there has been opportunity for compliance with such state or interstate order.
4. If state or interstate action to control such pollution is plain, speedy and efficient, the Attorney General shall not take action until the state or interstate case is disposed of.

Sections 8, 9, 10, 11 and 12 (Administration, Definitions, Other Authority, Separability, Short Title) were accepted.

* A bill before Congress of the United States to extend and strengthen the Water Pollution Control Act.

GARBAGE GRINDER EXPERIENCES, JASPER, INDIANA *

By Frank D. Wraight

Chief, Sewage Disposal Section, Indiana State Board of Health, Indianapolis, Ind.

The City of Jasper has a population of approximately 6,200 and is located in the southeast section of Indiana on the Patoka River. During prolonged dry periods, there is no flow of water in the Patoka River except that which is discharged from the sewage treatment plant. Most of the factories are dry industries with the exception of a poultry packing plant. Over 92 per cent of the residents own their homes which was one of the factors that made practical, community-wide installation of household garbage grinders. In 1949, Jasper was faced with abating pollution of the Patoka River and also finding a suitable method of disposing of garbage.

Because the garbage grinder would eliminate the garbage can and make possible a sanitary means of disposal at a reasonable cost, it was decided to proceed with the community-wide installation of the grinders and to construct a Sewage Treatment Plant designed to treat the waste from such installations. The Indiana State Legislature enacted a bill permitting the issuance of revenue bonds for the purpose of installing household garbage-grinders and providing the municipality with authority to establish such a project.

In March, 1950, the installation of garbage grinders was started and on August first there were approximately 730 grinders installed and garbage collection ceased. At the present time there are approximately 1,000 garbage grinders installed and practically all new houses are so equipped.

Inasmuch as there was no precedent for the design of the sewage treatment facilities, it was necessary to make some speculative estimates. Units which are affected principally by volume of flow were not increased over conventional design. The aeration tanks were increased by 50 per cent; the digester capacity was increased by 57 per cent; and the sludge drying bed area was increased by 60 per cent.

Table I shows how the average data for 1954 compares with the basis of design as well as the capacities of certain treatment plant units. The actual average flow is less than one-half the design flow and the population is a little more than one-half the design population.

Originally a 500 c.f.m. blower powered by a dual fuel gas-gasoline engine and an electrically driven 500

TABLE I.—Comparison of Operating and Design Data

Units	Data	
	Design	Operating (1954)
Population	10,500	6,200
Flows (m.g.d.)	1.0	0.460
Grit chamber		
Comminutor		
Sewage pumps [4] (g.p.m.)	2,200	
Raw sludge pump (g.p.m.)	100	
Return sludge pumps [2] (g.p.m.)	350	
Primary settling tanks [2] (surface rate, gal./sq. ft./day)	1,040	480
Aeration tanks [3] (detention time, hr.)	7.8	16.9
Blowers [3] (c.f.m.)	1,000	1,500
Final settling tanks [2] (surface rate, gal./sq. ft./day)	750	345
Digesters [2] (cu. ft./capita)	6.6	10.8
Sludge drying beds [8] (sq. ft./capita)	3.1	5.0

* Presented at 1955 Annual Meeting, Central States Sewage and Industrial Wastes Assn.; Rochester, Minn.; June 22–24, 1955.

TABLE II.—Effect of the Increasing Number of Grinders on B.O.D. and Suspended Solids

Grinders (no.)	Flow[1] (m.g.d.)	B.O.D. (p.p.m.)	B.O.D. (lb.)	Susp. Solids (p.p.m.)
0	0.407	151	510	114
850	0.502[2]	387	1,620	420
1,000	0.446	392	1,460	392

[1] Average flow.
[2] Increased flow due to rain.

c.f.m. blower were installed. The gas engine was operated on gasoline, as sludge gas was insufficient. As there were times during which the blowers were inadequate, the gas engine and blower were replaced with an electric motor and blower to make an additional 1,000 c.f.m. of air available at all times. This has solved the air supply problem.

Table II shows the effect of the number of garbage grinders on the B.O.D. and suspended solids of the raw sewage. Even though there were 150 more grinders in 1954 than in 1951, the total pounds of B.O.D. and suspended solids were slightly less. This is probably due to the fact that intense rain occurred during the 1951 survey and the solids which had accumulated in the lateral sewers were flushed out during the rain.

There was a significant increase in the raw sewage B.O.D. and suspended solids as a result of the increased use of residential garbage grinders (Table III). No attempt has been made to calculate a value which would show the per capita contribution of ground garbage. The data collected in wet weather indicates a flushing of deposits from the sewers.

The following treatment efficiency averages for 1954 indicate the satisfactory results obtained:

Item	Per Cent Removal	
	Primary Treatment	Over-all Treatment
B.O.D.	33	93.2
Susp. Solids	67.7	97.1

A comparatively high removal of suspended solids was particularly gratifying.

Table IV indicates the daily variation in mixed liquor dissolved oxygen. Usual operation requires that only the 1,000 c.f.m. blower be operated during the weekend with the addition of the 500 c.f.m. blower during the week. This operation has maintained the dissolved oxygen content within reasonable limits.

Prior to May, 1955 all three aerators were in use. Based on aerator volume the pounds of B.O.D. removed per thousand cubic feet was rather low, approximately 16 lb. per 1,000 cu. ft. Since the first part of May, 1955, only two aerators have been in use with no apparent loss in the over-all removal of B.O.D. and suspended solids. Mixed liquor solids average 2,500

TABLE III.—Per Capita Waste Discharge

Month	Grinders (no.)	Determination (lb. per day per capita)				
		B.O.D.	Total Solids	Total Vol. Solids	Susp. Solids	Susp. Vol. Solids
1950						
Mar.	80	0.12	0.39	0.18	0.19	0.14
Oct.	785	0.18	0.41	0.26	0.20	0.16
1951						
May	850	0.31	0.88	0.59	0.34	0.29
1954						
Oct.	1,000	0.32	—	—	0.34	—

p.p.m., which is higher than carried previously. During this same month 23.5 lb. of B.O.D. were removed per 1,000 cu. ft. of aerator tank volume.

Unfortunately, few data are available which would indicate the quantity and quality of the sludge pumped to the digester. However, based on the suspended solids removed and allowing a 20 per cent loss in the secondary process, it has been calculated that 1 lb. of solids produces from 8 to 10 cu. ft. of gas.

After treatment plant operation began, it was discovered that it was impossible to use both return sludge pumps and maintain the desired solids concentration in the mixed liquor. The inability to return sludge was due to the fact that the sludge division box was installed at a lower elevation than specified in the design, resulting in a bottleneck in the return sludge system. The box was raised approximately 3 ft. solving this problem.

The sludge index and mixed liquor solids curves (Figure 1) illustrate the conditions existing prior to raising the division box. These curves are based on Professor Bloodgood's statement that the sludge index converted to

TABLE IV.—Daily Variations in Dissolved Oxygen

Days	Dissolved Oxygen (p.p.m.)		
	Min.	Av.	Max.
Mon.	1.1	1.2	1.6
Tue.	1.4	2.3	3.3
Wed.	1.3	2.9	4.7
Thurs.	1.1	1.6	3.6
Fri.	1.1	2.1	4.3
Sat.	1.4	2.6	3.6
Sun.[1]	—	—	—

[1] No determinations made.

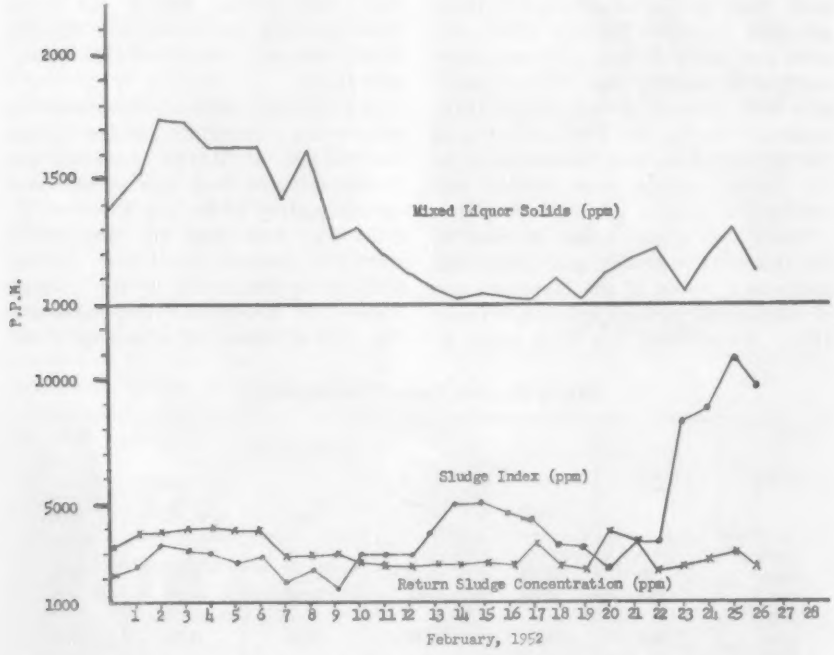

FIGURE 1.—Sludge index and mixed liquor solids prior to increasing the return sludge volume.

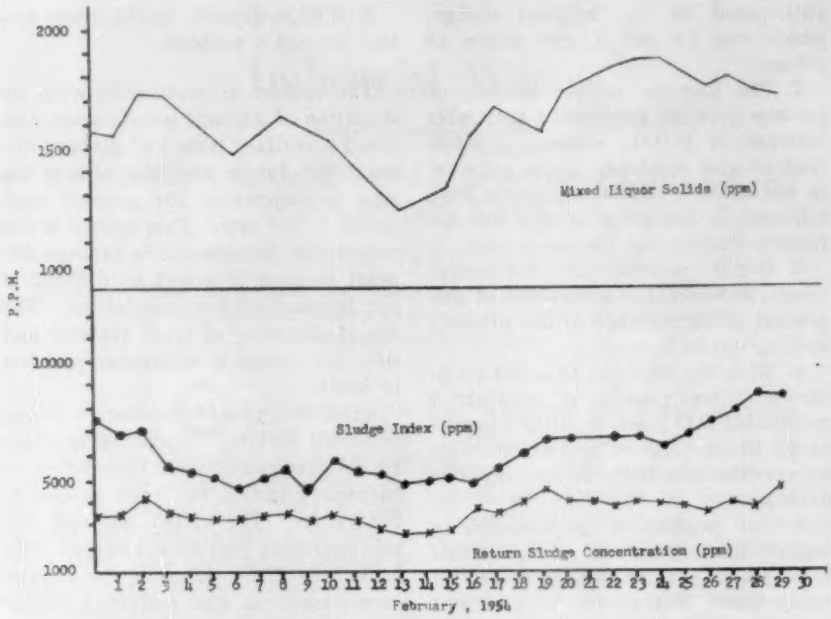

FIGURE 2.—Sludge index and mixed liquor solids after increasing the return sludge volume.

parts per million should be equal to or higher than the return sludge concentration. During this period of operation considerable difficulty was occasionally encountered in producing a settleable sludge.

After eliminating the return sludge bottleneck, it was possible to operate under the conditions shown in Figure 2. The mixed liquor solids increased toward 2,000 p.p.m. and the sludge index improved.

Sewer System

The Jasper sewer system contains approximately 15 miles of combined and sanitary sewers. New sewers are designed for sanitary flow. However, the original system, which was constructed in 1910 and expanded after 1925, is a combined system. Grades vary from 0.16 per cent for the 12-in. trunk sewers to 7.25 per cent on the 6-in. lateral sewers. Considerable building is now taking place north and east of the city, and a large area recently has been annexed which will add several hundred residences to the sewer system.

Before the garbage grinder project was initiated several manholes were selected as check points to observe the sewer outlets from various sections of the city. The purpose was to determine whether or not sludge deposits would increase with an increase in garbage grinders.

A recent inspection of the outlet of an 8-in. sewer on a .61 per cent grade, with a theoretical velocity of 2.5 ft. per second flowing full, revealed no excessive sludge deposits. Originally there were 49 garbage grinders on this lateral; now there are 70 to 80 units. Principal sewer maintenance problems are tree roots and infiltration.

Summary and Conclusion

1. Air requirements per pound of B.O.D. removed are more than were

anticipated in the original design, which was 1.4 cu. ft. per gallon of sewage.

2. The increase in the number of garbage grinders produced a noticeable increase in B.O.D., suspended solids content and volatiles. Data collected in wet weather indicates deposits have collected in the sewer system but are readily flushed out during a rain.

3. B.O.D. removals are not exceptional; however, the percentage of suspended solids removed in the primary settling tanks is good.

4. With the addition of a 500 c.f.m. blower it was possible to maintain a reasonable D.O. content in the aerators at all times. It now appears possible to operate effectively using only two aerators.

5. Gas production is considerably higher than found in sewage treatment plants not handling garbage. In many cases it may be 80 per cent higher.

6. Sludge deposits in the sewer system are not a problem.

The system of community-wide installation of garbage grinders has provided a sanitary means of garbage disposal for Jasper and has been a big step in improving the general sanitation in the city. This system is not considered the ultimate in garbage disposal because it would be difficult if not impractical for some cities. The job of disposing of trash remains and this can create a sanitation problem in itself.

After five years of operation it can be stated that the disposal of garbage by the community-wide installation of garbage grinders has been successful for Jasper. The system has had certain operating problems; however, they have been little different from those encountered at any activated sludge sewage treatment plant.

AMERICAN SANITARY ENGINEERING INTERSOCIETY BOARD

The certification plan for sanitary engineers recently became an incorporated entity. The American Sanitary Engineering Intersociety Board has been organized and incorporated "to improve the practice, elevate the standards and advance the cause of sanitary engineering; to grant and issue to engineers, duly licensed by law to practice engineering, certificates of special knowledge in sanitary engineering or in any field thereof."

The new group has been authorized and underwritten by the American Society of Civil Engineers. Organizers of the Board included representatives of the American Society of Engineering Education, American Society of Civil Engineers, American Public Health Association, American Water Works Association and the Federation of Sewage and Industrial Wastes Associations. A headquarters has been opened in the Engineering Societies Building, 33 West 39th St., New York City. Officers of the Board are: Professor Earnest Boyce, Chairman; W. A. Hardenbergh, Vice-Chairman; R. S. Rankin, Treasurer, and Francis B. Elder, Secretary.

First step will be the development of the certification process by a special committee under the chairmanship of R. F. Faust, AWWA. Invitations to membership will be offered to mechanical and chemical engineers with appropriate experience, in addition to members of the civil engineering profession.

Industrial Wastes

SOLVING A LARGE-SCALE WASTE PROBLEM AT APPLIANCE PARK *

By Kenneth S. Watson and C. M. Fair

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Major appliances of General Electric Company were being manufactured in a number of plants in various part of the country when the decision was reached in 1949 to consolidate the manufacturing operations in a single new plant to be located in Louisville, Kentucky. Consistent with the usual policy of the company, it was agreed that approved waste treatment would be provided as a part of the new plant.

The new plant, which is the third General Electric facility to be located on a park-like campus, is called Appliance Park. Construction was started in September, 1951 and at present five major buildings containing about 3,700,000 sq. ft. of floor space have been virtually completed. A sixth major building which will contain about 300,000 sq. ft. of additional floor space has recently been authorized. Figure 1 is an aerial photograph of the plant site which covers almost 1,000 acres.

For the purposes of this paper the discussion of waste facilities is broken down into the definition of the problem, the collection system, the waste treatment plant, and the general costs involved in constructing and operating these treatment facilities.

* Presented at 28th Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Atlantic City, N. J.; Oct. 10–13, 1955.

Defining the Waste Problem

When the building of a new manufacturing plant was planned, a program was launched to determine the quantity and character of the wastes leaving each appliance manufacturing operation scheduled to be moved to Louisville. Study also was made of the amount of water being used in these operations and the possibility of using water management to reduce water requirements and the volume of waste which would need to be treated. Finally, the various loads obtained were totaled and adjusted to cover the increased production capacity anticipated at the new plant.

After the decision had been reached that pretreated wastes from the plant would be discharged into the Louisville Metropolitan Sewer System, a series of conferences were held with Metropolitan Commission and Kentucky Department of Health officials. As a result of these conferences agreement was ultimately reached concerning the concentration limits which the wastes should meet prior to discharge into the Metropolitan Sanitary District.

In considering waste treatment for this plant, treatment facilities were planned with maximum flexibility. This philosophy seemed sound since the definition of the waste load to be expected was so complicated that the

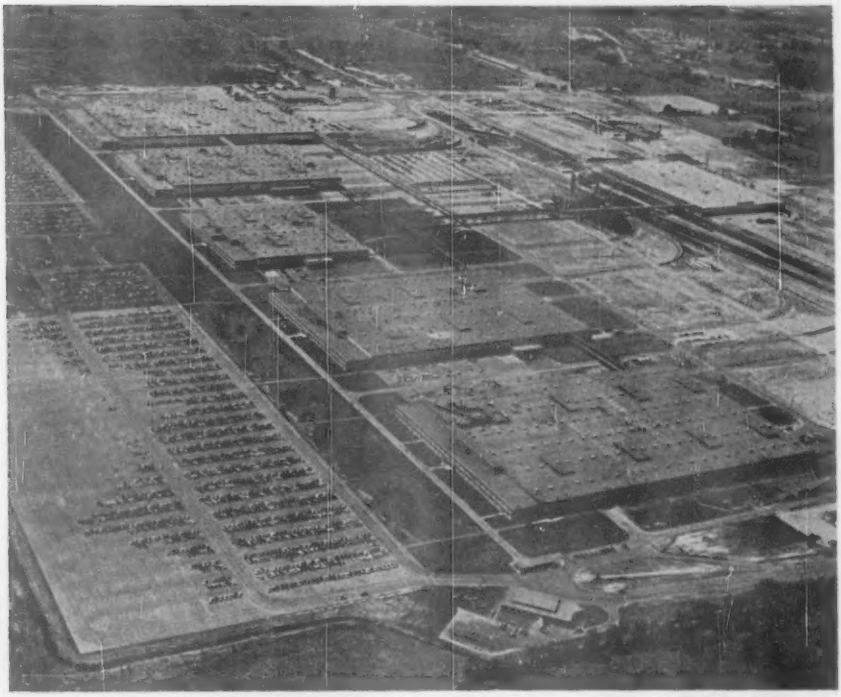

FIGURE 1.—Principal manufacturing buildings at Appliance Park, General Electric Company, Louisville, Ky.

final result could not be completely accurate. It also appeared that in connection with the provision of sanitary sewage treatment facilities for the Metropolitan Sanitary District, the industrial waste requirements might be modified. For these reasons, a plant with sufficient flexibility to meet changing requirements appeared necessary.

The industrial wastes at Appliance Park originate in the pickling, plating, bonderizing, anodizing and other metal finishing operations. The survey work on the magnitude and scope of the waste problem to be expected indicated that sufficient acid, cyanide and chromium would be discharged from these metal finishing operations to require treatment for each type of wastes. Furthermore, it was apparent that there would not be enough metal ions with the exception of chromium to cause any problem.

From the results of the problem definition survey, it was decided to design the treatment plant for continuous operation. It was decided further to make maximum use of the holding and equalization capacity of the waste collection system and to keep the detention time of the receiving tank to less than one hour.

The next decision was to design the treatment plant to handle 60 per cent of the estimated ultimate volume. The plant was to be constructed with parallel tanks so that other treatment units could be added if the ultimate waste load exceeded the reduced volume for which the plant was designed.

Another reason for cutting the initial capacity of the waste treatment plant was that water conservation or management was being incorporated into the new plant in the form of a mill water recirculation system. This

mill water system (Figure 2) consists of a spray pond and the necessary pumps so the cooling water at the park can be used repeatedly. This system was installed to provide fire protection and to reduce city water requirements, thereby keeping costs and the volume of industrial wastes to a minimum.

Collection Systems

There are a total of seven types of sewers at Appliance Park handling sanitary sewage, storm water, combination storm water and mill water return, oily wastes, acid and alkali rinse water, chromium rinse waste and cyanide rinse waste. The waste collection system (Figure 3) comprising the last three sewers listed conveys the wastes to the industrial waste treatment plant. The manufacturing buildings are underlaid alternately at 40-ft. intervals with combination storm and mill water return sewers and acid-alkali rinse waste drains. Sanitary and oily waste sewers have been installed only where needed at specific points. Chrome and cyanide rinse wastes sumps have been spotted at points of waste production. The latter wastes are then pumped through overhead lines into covered basins outside the buildings, from which flow is by separate underground gravity lines to the industrial waste treatment plant.

There are about 39 miles of sewer lines consisting of 15 miles for storm and mill water return, 7 miles for sanitary purposes and 17 miles for industrial waste uses. All sewer lines, except storm sewers, are vitrified tile. The lines carrying industrial wastes have been jointed with acid proof mastic. Acid-alkali rinse waste mains are 36 in. in diameter. Storm sewer mains are 2- to 9-ft. diameter reinforced concrete.

Other waste collection facilities, particularly for handling spent pickling acids, have been provided in addition to the sewers mentioned. Acid holding tanks are available at various points from which spent acids can be equalized into the acid-alkali line or used for water conditioning in the mill water system. A 2,500 gal. saran-lined steel tank trailer is also available to receive spent acid from process tanks situated near an outside wall of a building, or from a holding tank. The acid pumped into the tank trailer is hauled to the treatment plant where it is placed in a holding tank to be used for waste treatment purposes.

The wastes collection system is somewhat unique because it serves a "park" type plant. Since the wastes

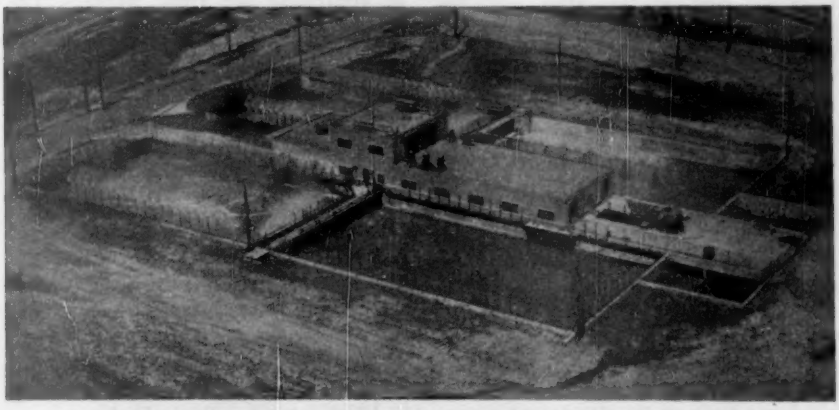

FIGURE 2.—Mill water reservoirs and pump house utilized in the water management program.

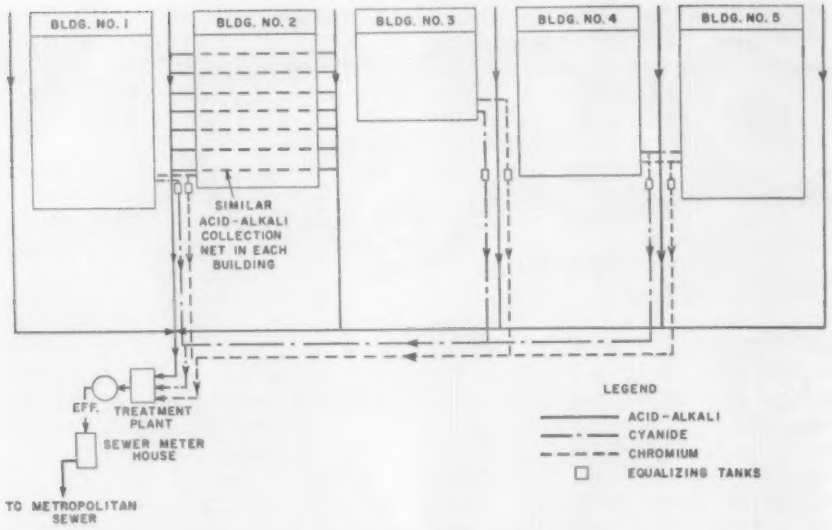

FIGURE 3.—Schematic layout of the industrial wastes collection system.

from the entire park are treated in a common waste treatment plant, the sewer lengths in the collection system are much longer than is the case with many factories.

The Treatment Plant

The principle of waste treatment conceived and built into this manufacturing plant is that of first conditioning the separate rinse waters carrying acids and alkalies, cyanides and chromium. Following this step, all three types of wastes are combined in a final equalization and adjustment basin before being discharged to the sanitary district sewer. Figure 4 is a flow diagram of the treatment plant processes. In order to furnish a more complete description of the equipment and treatment in each process, each treatment line will be considered in some detail.

Acids and Alkalies

The acid-alkali wastes total about 3.7 m.g.d. and flow into a 50,000 gal. receiving tank at the treatment plant. This storage capacity can be increased to about 150,000 gal. by flooding the influent sewers to the overflow level of the holding tank. The tank has a plastic lining and an acid brick bottom.

Four 1,100 g.p.m. pumps take suction from this receiving tank and discharge through an 18-in. saran-lined header to a 20,800-gal. waste mix basin. Lime slurry, as a general rule, or occasionally acid is added in this basin to neutralize the rinse water. The neutralized waste then flows to a final equalization basin prior to discharge to the interceptor sewer of the sanitary district.

Hydrated lime is fed in slurry form to the agitated neutralization basin at an average rate of 3,000 lb. per day, using two 300 lb. per hour feeders. The rinse water has an average pH slightly below 3 when received at the treatment plant and is raised by the neutralization operation to a pH of 6.

Cyanide Rinses

The cyanide rinse water, amounting to 128,000 g.p.d. (Table I), has a cyanide concentration between 10 and 222

TABLE I.—Volume of Wastes by Type, Jan.-June, 1955

Wastes	Volume (g.p.d.)		
	Average	Maximum	Minimum
Acid-alkaline rinse	2,886,000	4,087,000	1,564,000
Chromium rinse	56,400	97,000	15,000
Cyanide rinse	128,600	239,000	43,000
Total industrial waste	3,071,000	4,388,000	1,623,000
Sanitary sewage	295,000	356,000	256,000
Total wastes	3,366,000	4,744,000	1,879,000

p.p.m. This flow is received in a 5,000 gal. holding tank at the waste treatment plant from which it is pumped into the first of three 20,000-gal. treatment basins in series. Caustic soda and chlorine are fed into these agitated reaction tanks to raise the pH to 10 to 10.5 and oxidize the cyanide.

It requires about 400 lb. of chlorine per day to do the oxidation job. This represents the feeding of chlorine on the basis of about 7 lb. of chlorine to 1 lb. of cyanide radical. Cyanide sludges from heat treatment operations or plating tank cleaning are sometimes hauled to the treatment plant and slowly fed into the system.

Chromium Rinses

The rinse water from the chromium system averages about 56,000 g.p.d. It has a hexavalent chromium concentration ranging between 2.2 and 64 p.p.m. and flows into a 5,000 gal. acid proofed holding basin. This rinse water then is pumped into a 17,000 gal. acid proofed reaction tank where waste acid is added to reduce the pH to 3. Sulfur dioxide, in the ratio of 1 lb. of gas to 1 lb. of chromium, is then fed from a 500 lb. per day feeder to reduce the chromium to the trivalent form.

It has been necessary thus far to treat the chromium wastes only occasionally as a result of lower concentrations and smaller volumes than originally anticipated. As a result of the recent expansion in the existing manufacturing operations and the addition of new ones an increase in the chromium load can be anticipated in the near future.

Common Facilities

In addition to the items already considered for handling the three types

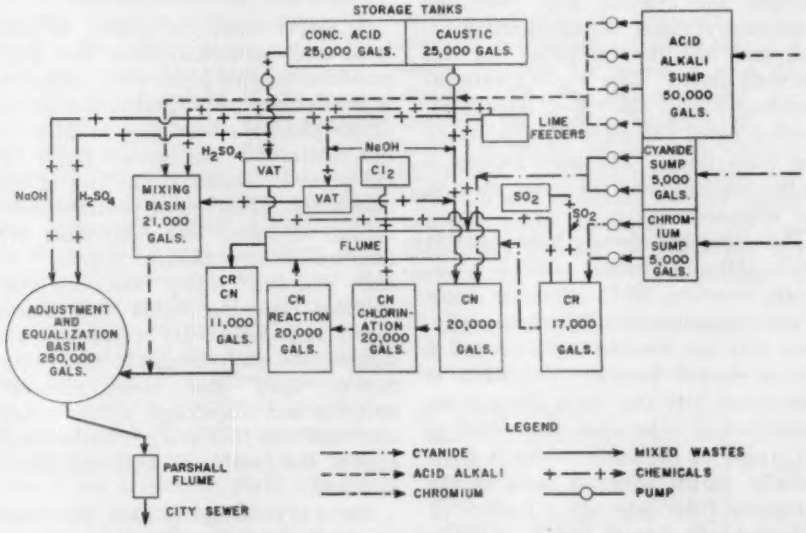

FIGURE 4.—Flow diagram of the industrial waste treatment plant, General Electric Company, Louisville, Ky.

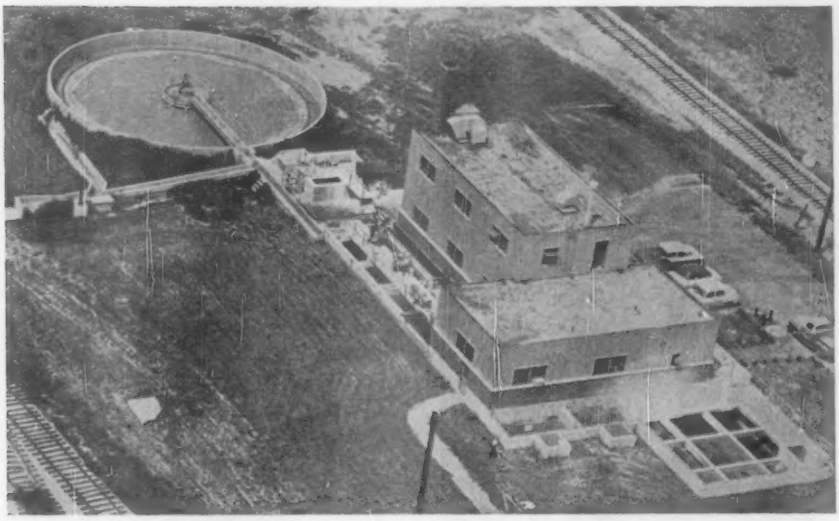

FIGURE 5.—Industrial waste treatment facilities.

of industrial wastes, a number of facilities have been provided at the waste treatment plant to facilitate the treatment of all the wastes. Figure 5 is an aerial photograph showing the waste treatment building and the treatment tanks clustered around it. These units include the cyanide and chromium treatment systems, the equalization basin, and the flow recorder for the parshall flume. The waste treatment building houses the office, laboratory, control panel and chemical feeders on the main floor. The pipe gallery is in the basement and the pump room in the sub-basement.

Reference has already been made to the equalization basin. This is a circular, concrete, 60-ft. diameter basin of approximately 250,000 gal. capacity. This unit has a center agitator and a conical shaped bottom. The waste is introduced into the basin through an underground pipe that discharges at the center of the basin where it flows radially to the overflow weir at the periphery. An interesting feature of this basin is that it can be readily modified into a clarifier to remove solids should this step become necessary.

As mentioned previously, all the industrial wastes are combined in this basin where the final pH or other adjustments can be accomplished.

Controls and Instrumentation

A rather extensive system of controls and instrumentation has been provided for this plant since operation is continuous and equalization ahead of the plant has been kept to an absolute minimum. The control panel for the plant is located at one end of the laboratory. This panel contains indicating, recording and controlling pH and oxidation-reduction meters. It also contains pump and agitator switches, running lights and alarm lights and bells. By means of air signals, the controlling pH and redox meters open and close pressure switches and diaphragm valves which start and stop lime and gas feeders and control the feeding of acid and alkali solutions.

Aorta type samplers have also been provided which are automatic and proportioned by flow meters. The chlo-

rine and sulfur dioxide drum scales are equipped with recorders for weight loss records. Caustic and spent acid solutions are proportioned by automatic valves from head tanks filled by float switch controlled pumps.

Operation and Results

Since the waste treatment plant is of the continuous type with minimum holding capacity before treatment, it is operated on a 21-hr. day. Two men, a treatment plant operator, and a laboratory technician are on duty during each operating shift. In addition, a maintenance man is on duty during the day shift. Since the factory normally operates two shifts, the treatment plant is kept in operation for 21 hr. by staggering the reporting times of the personnel. A schedule of continuous sampling and analyzing is maintained to insure that the treatment plant is operating properly. As a further duty the mill, boiler and condensate waters are analyzed, and control determinations are made on the coal delivered to the manufacturing plant. Continuous records are kept of the operation of the waste treatment plant and the chemicals required in the treatment processes.

The supervisor has overall responsibility for the operation of the treatment plant. In addition, he is responsible for a plant-wide waste control and water management program. In this program the supervisor and the other personnel spend considerable time on educating manufacturing personnel on the part they must play in making waste control and treatment operations function properly.

An evaluation of the type of wastes which the plant will deliver indicates that it is doing a completely satisfactory job. Table II shows the concentration of various materials in a composite of the treatment plant effluent. The average hexavalent chromium concentration in the effluent is 0.78 p.p.m. and the cyanide is 0.19 p.p.m. A brief review of Table II will indicate that the effluent being discharged by the treatment plant should be acceptable to most municipal sewer systems.

TABLE II.—Effluent Analysis, Jan.-June, 1955

Determination	Average	Maximum	Minimum
pH	7.34	9.4	3.2[1]
Chromium (Hex) (p.p.m.)	0.78	3.05	0
Chromium (total) (p.p.m.)	1.64	9.00[2]	0.07
Cyanide (p.p.m.)	0.19	15.0[3]	0
Iron (p.p.m.)	13.88	28.6	1.0
Copper (p.p.m.)	0.83	4.42[4]	0.08
Susp. solids (p.p.m.)	198.3	487.0	25.0
Dissol. solids (p.p.m.)	598.8	1309.0	396.0
Total solids (p.p.m.)	766.8	1510.0	564.0
Chlorine demand (p.p.m.)	4.0	6.0	1.5

[1] Less than pH 5.5 only once.
[2] Above 5.0 p.p.m. only twice.
[3] Above 2.0 p.p.m. only once.
[4] Above 3.0 p.p.m. only once.

Considerable quantities of chemicals are required in the operation of the treatment plant. Table III lists the quantities consumed for the first six months of 1955. Lime (186 tons) and chlorine (30 tons) are the principal chemicals used.

The operation of the waste treatment plant over the 18-month period since its completion indicates that it is doing the job for which it was designed. The treatment plant has reached the design capacity of 4.5 m.g.d. as shown by the flow volumes in Table I. Since some additional construction and expanded production of manufacturing facilities is contemplated, an increase in waste load of perhaps 20 per cent is expected by

TABLE III.—Chemicals Consumed, Jan.-June, 1955

Chemicals	Monthly Average (lb.)	Total (lb.)
Lime	62,100	373,000
Caustic soda	7,633	45,976
Chlorine	10,093	60,558
Calcium hypochlorite	617	3,700
Sulfur dioxide	64	384

TABLE IV.—Waste Treatment Plant Operating Costs,* Jan.-June, 1955

Item	Costs	Per Cent
Payroll	$16,564	25
Chemicals	11,905	18
Supplies and Services	5,232	8
Depreciation and Taxes	32,328	49
Total	66,029	100

* Wastes treated, 441 m.g. at $0.15 per 1,000 gal.

the end of 1956. To meet this expanded load one additional acid-alkali pump is now on order and additional parallel basins are contemplated.

Operating personnel now have acquired sufficient actual experience to operate the plant on a completely sound basis and effort is being made continuously to improve operating processes and efficiencies. Among the improvements considered are: ion exchange recovery of chromium, more economical neutralization chemicals, more economical chemical handling systems and an increase in efficiency in the treatment of cyanides. In addition to the improvements already mentioned, the possibility of re-using the effluent from the waste treatment plant is also under consideration.

Installation and Operating Costs

The total cost of the waste treatment facilities exclusive of the collection system is approximately $900,000. At the present capacity of 4.5 m.g.d. the installation cost amounts to almost $200,000 per million gallons of daily capacity. As a result of the flexibility concept employed in the design and construction of the treatment plant, it is believed that the capacity of the plant can be increased to 7 m.g.d without excessive additional costs. If this is done the expanded plant would have a cost of approximately $135,000 per million gallons of daily capacity.

The operating costs for the first six months of 1955 amounted to $66,029 and are summarized in Table IV. Almost one-half the operating cost is in depreciation and taxes and the remainder is equally distributed between payroll, supplies and services.

Table V shows the cost per 1,000 gal. for treating each of the industrial wastes. It is interesting to note that the cost of treating the total waste is about $0.15 per 1,000 gals. The wastes vary in treatment costs per 1,000 gal. as follows: $0.106 for acid-alkali wastes; $0.90 for cyanide rinses; and $0.41 for chromium rinses.

Summary and Conclusions

As a result of being able to look in retrospect at a period of four years since waste treatment was first considered for Appliance Park and a period of 18 months of actual waste treatment experience, conclusions can be drawn as follows:

1. The industrial waste treatment plant at Appliance Park, during its 18 months of operation has treated the

TABLE V.—Volume and Treatment Costs for Each Type of Waste, Jan.-June, 1955

Waste	Volume		Treatment Costs[1]		
	Gal. per day	Per Cent	$ (6 mos.)	Per Cent	$/1,000 gal.
Acid-alkaline rinses	2,886,000	94.0	43,000	65	0.106
Chromium rinses	56,400	1.9	3,300	5	0.41
Cyanide rinses	128,600	4.1	16,500	25	0.90
Spent acids	—	—	3,229	5	—
Total (average)	3,071,000	100.0	66,029	100.0	15.0

[1] Estimated.

complex metal finishing wastes in such a manner and to such a degree as to discharge a satisfactory effluent into the Metropolitan District Sewer System. Familiarity with operations contemplated at Appliance Park indicates that the treatment facilities are sufficiently flexible so that they will be adequate with minor additions to do a completely satisfactory job for a long time to come.

2. The success of the procedure used at Louisville of early consultation with representatives of the Metropolitan Commission and the State Department of Health concerning the degree of treatment desirable for this new manufacturing facility attests to the fact that such a cooperative approach to the problem is the best one.

3. Experience with the waste treatment problem to date at Louisville indicates that the procedure of reducing the volume of wastes by water management and of providing a minimum functional plant which can be expanded or modified readily to correspond to the waste load is a sound approach for a manufacturing facility as large and complex as Appliance Park.

4. The waste treatment plant at Louisville had an installation cost of approximately $900,000. With an average waste load of 3.071 m.g.d., typical of operations during the first half of 1955, the treatment facilities would have an operating cost of $132,000 per year.

5. The installation and operating costs of the waste treatment facilities indicate again that all reasonable effort to reduce the waste treatment problem by steps short of treatment is justified since the capital investment and operating costs must be passed along through increased product prices to purchasers of the product manufactured.

6. The collection system at Appliance Park is longer and more complex than would be required for many factories. This is true because the manufacturing plant is of the park type with buildings well separated to provide ample space for expansion and because all wastes are treated in a common treatment plant.

GRADUATE ASSISTANTSHIP

Georgia Institute of Technology, School of Civil Engineering, has a graduate research position available for study on the treatment and disposal of radioactive sludges. The salary is $2,000 per year on a part-time basis and opportunity is available for advance study, either M.S. or Ph.D.

Further information may be obtained from Dr. Werner N. Grune, School of Civil Engineering, Georgia Institute of Technology, Atlanta, Ga.

BIOLOGICAL TREATMENT OF PETROLEUM WASTES

By Robert A. Baker and Roy F. Weston

Associate Chemical Engineer, and Sanitary Engineer, Respectively, Atlantic Refining Co., Philadelphia, Pa.*

I. Biological Filter Studies

Recent years have witnessed the advent of an active water resources conservation program. No longer are streams used as open sluices for the discharge of sanitary and industrial wastes. Each industry is faced with the need for discharging an effluent of a quality satisfactory for propagation of aquatic life and utilization by downstream consumers.

Because of the diversity of operations within the limits of the typical oil refinery, a waste water problem exists which is unlike that common to other industries. Waste components may include oils, waxes, phenolates, naphthenates, sulphonates, polysulfides, mercaptans, caustics, acids and many other organic and inorganic contaminants.

Because of the diversity of these pollutants, no specific treatment scheme may be applied as a universal cure-all. The result is the application of pretreatment by separation, skimming and oxidation, as well as isolation of specific sources prior to chemical or biological treatment.

The Point Breeze Refinery of the Atlantic Refining Company is situated on the Schuylkill River in Philadelphia, Pa. Earlier investigations showed that because of the processing conducted in the two principal manufacturing areas, the wastes were different.

The wastes from one area were treatable by chemical flocculation (1) while the wastes from the other area may possibly be adaptable to biological treatment. After the feasibility of biological treatment was ascertained by bench-scale experiments, a laboratory scale biological filter was used to obtain pilot plant data. These studies were reported previously (2).

Surveys of the south yard wastes showed the presence of specific contaminants for which the effects on biological filters were unknown. Since some of these materials are produced intermittently, sudden loading could upset filter operation and cause poor effluent. Therefore, pilot plant investigations were made to determine the effects of these wastes.

Effect of Lead

Lead may originate in refinery waste waters from the cleaning of gasoline storage tanks and litharge washings. Bioassay tests indicated that the 24-hr. median tolerance limit of lead at 20° C. was 1.95 p.p.m. for Blue Gill Sunfish (3). It was also desirable to determine what levels of lead might cause deleterious effects on trickling filters. Each pilot plant trickling filter, shown schematically in Figure 1, consisted of three 6-in. diameter by 26-in. long terra cotta pipe sections supported vertically in series. The sections were spaced about 1 in. apart, with perforated plates inserted to assure even distribution of the flow. The column packing was trappe rock with an average diameter of 1 in. and an effective packing depth of 6 ft.

The influent was contained in four

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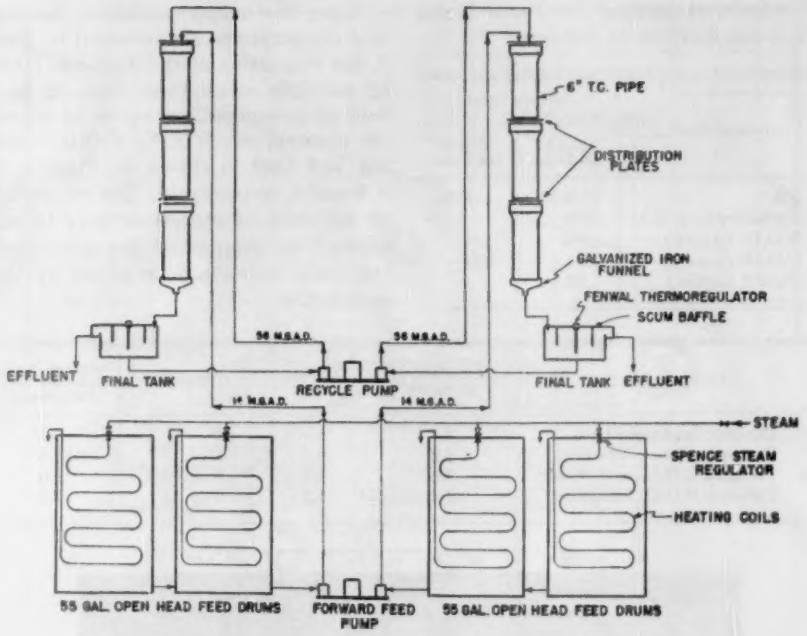

FIGURE 1.—Flow diagram of the biological filter pilot plant for treating petroleum wastes.

55-gal. barrels (Figure 2), the interior of which had been painted with Ucilon to retard corrosion and prevent bare metal contact. Tygon tubing was used for carrier lines as this facilitated cleaning and prevented absorption of materials such as phenols.

The filters were seeded with a synthetic sludge prepared by aerating Schuylkill River water, milk and soap. Then, the filters were operated on gradually increased loading of synthetic influent. After sufficient constancy of operation was obtained at the desired loading-levels, the tests began.

The great variation in the composition of refinery wastes necessitated that a synthetic waste be used, thus limiting the number of possible errors. This synthetic influent consisted of Schuylkill River water, waste caustic from kerosene treatment and dried milk. Sulfuric acid was used for pH adjustment and lead was added as lead acetate during initial lead study. The B.O.D. contributed by the lead acetate was negligible.

The influent flow rate was maintained at 14 m.g.a.d. A total hydraulic flow rate of 79 m.g.a.d. was obtained with a recycle rate of 4:1.

The synthetic influent was prepared daily since each 24-hr. period was considered a single test run. One set of samples was analyzed daily from each filter. The settling tanks, feed tanks and tygon tubing were cleaned and flushed every day to prevent the accumulation of biological growth on septic floc.

The lead study was made in two parts. Part I consisted of 96 test runs at widely varying influent temperatures and lead loadings up to 18 p.p.m. Part II consisted of 84 test runs at $90 \pm 2°$ F. and lead loadings up to 80 p.p.m. The influent temperature was controlled by steam-heated coils.

The influent loading for Parts I and II is summarized as follows:

Influent Conc.	Median Values	
	Part I (96 Tests)	Part II (84 Tests)
pH	6.60	6.32
Turbidity (p.p.m.)	186	—
B.O.D. (p.p.m.)	164	116
C.O.D. (p.p.m.)	294	222
Phenol (p.p.m.)	22.4	—

Since two major variables, loading and temperature, are involved in Part I, the evaluation of the data was made by multiple correlation. The effect of lead at concentrations up to 18 p.p.m. on removal of B.O.D., C.O.D., phenol and lead is shown in Figures 3, 4, 5 and 6, respectively. The reliability of the lines of regression may be examined by comparing the individual test runs with those predicted by the correlation:

Effect of Lead on	Median Deviation	Average Error	Range	Standard Deviation from Line of Regression
Per cent lead reduction	2.4	4.4	0 ± 27.5	6.4
Per cent phenol reduction	6.2	9.1	0 ± 32.7	11.6
Per cent C.O.D. reduction	2.5	4.2	0 ± 17.7	5.6
Per cent B.O.D. reduction	2.8	4.5	0 ± 18.7	6.0

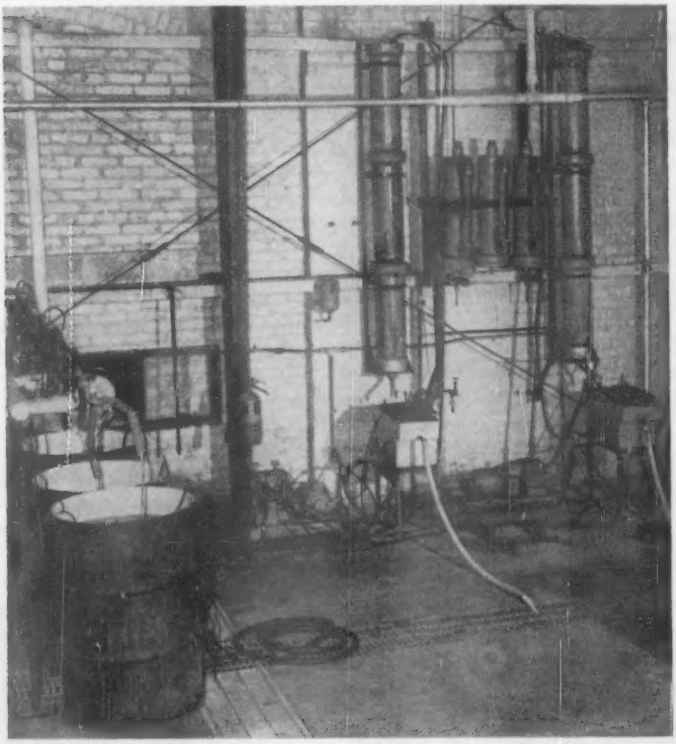

FIGURE 2.—Pilot-plant trickling filter.

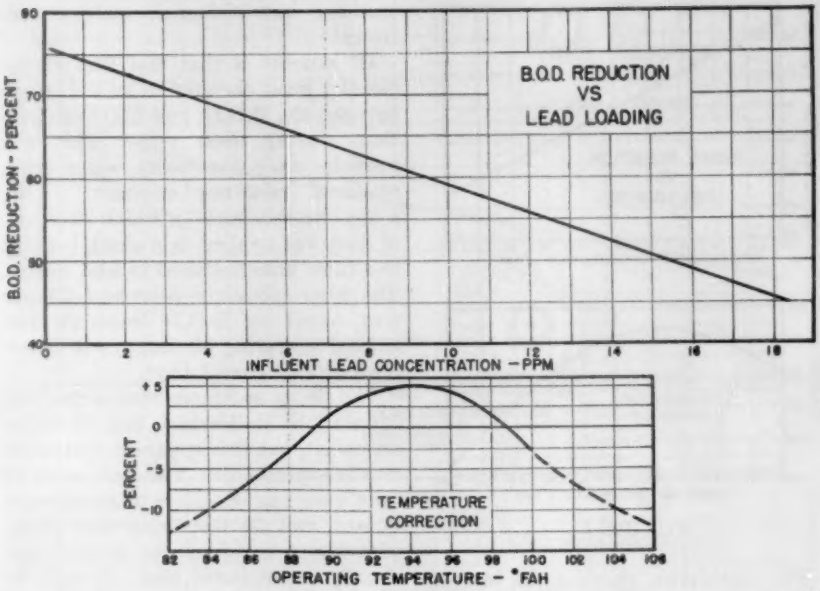

FIGURE 3.

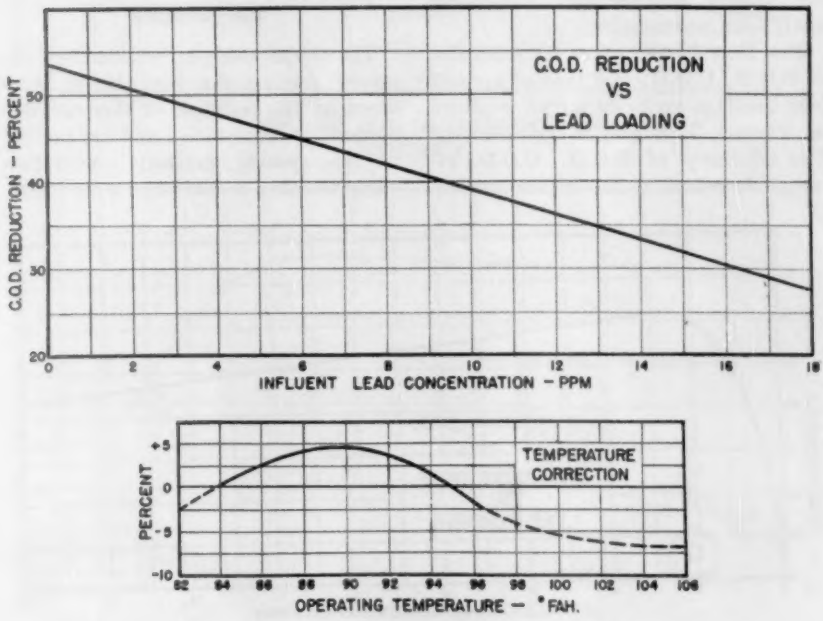

FIGURE 4.

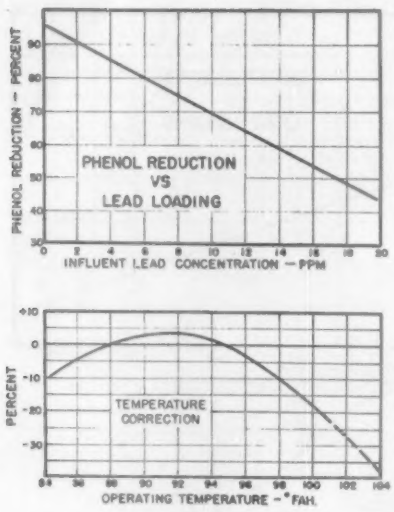

FIGURE 5.

The correlation showing the largest deviation is that of "per cent phenol reduction" at varying lead loading. A factor influencing this result is the low phenol loading and subsequent analytical inaccuracies.

For Part II the effect on reduction of B.O.D., C.O.D., and lead at applied lead loadings up to 80 p.p.m. is shown in Figures 7, 8 and 9, respectively. The efficiency of B.O.D.—C.O.D. removal decreased at the higher concentrations, but biological activity continued.

Of interest is that fact that exceptionally good correlation was obtained between the B.O.D. and C.O.D. reductions during these tests. This was possible since the waste composition remained relatively constant. Also, a test run was made in which 30 p.p.m. of lead was applied as a shock loading to a filter unaccustomed to lead wastes. The filter efficiency decreased 20 per cent based on B.O.D. removal, but normal operating efficiency was recovered within several days.

The slopes and intercepts of the lead correlations in Parts I and II differ somewhat, but this apparent contradiction is explainable. The first series of tests were run on filters unaccustomed to lead, and the later runs were made with filters in which the zooglea had already accumulated lead. It may be concluded from this study that lead did not destroy biological activity; however, efficiency was impaired.

Temperature

The temperature correction plots obtained during the lead study were based on the isolation of this variable by multiple correlation and are limited to the specific primary correlation when treating a similarly toxic waste.

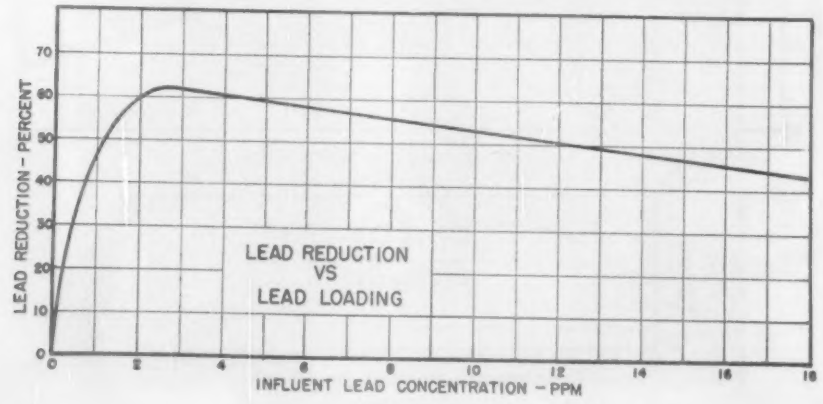

FIGURE 6.

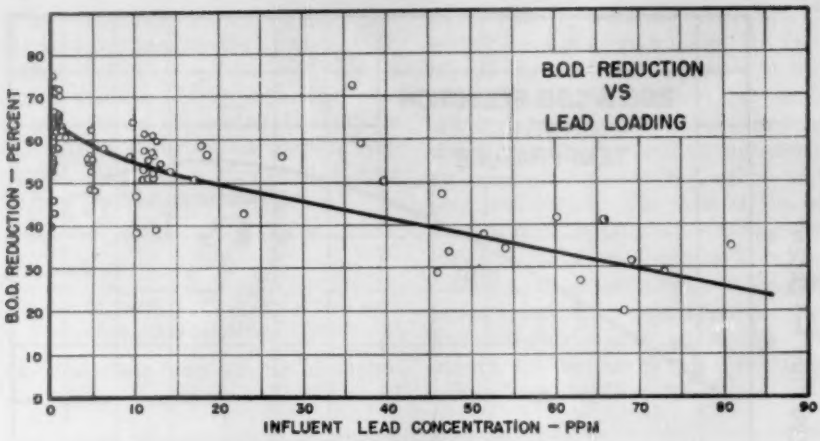

FIGURE 7.

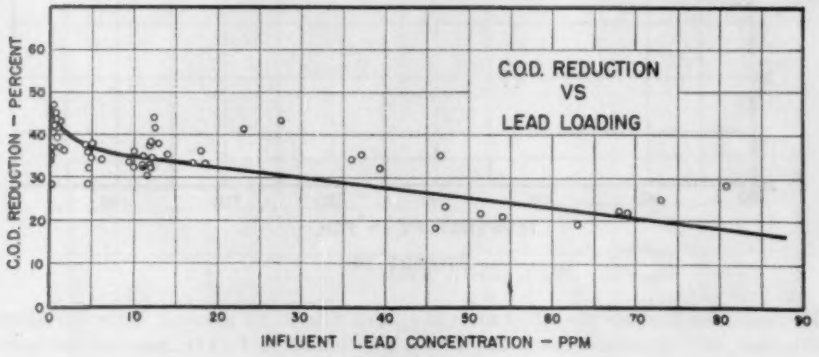

FIGURE 8.

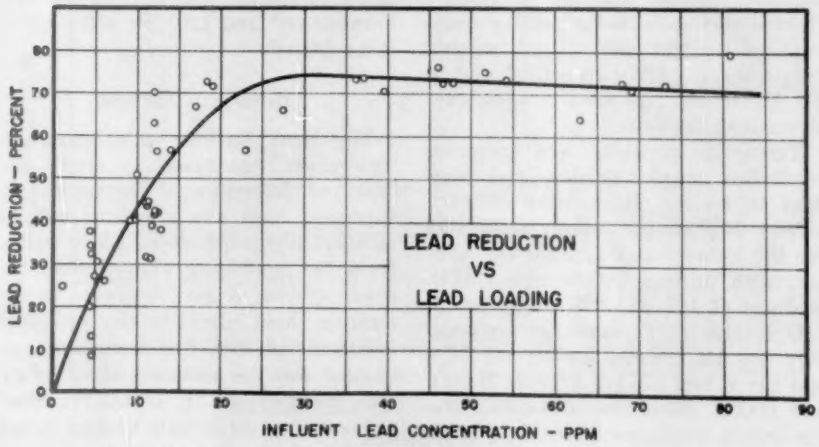

FIGURE 9.

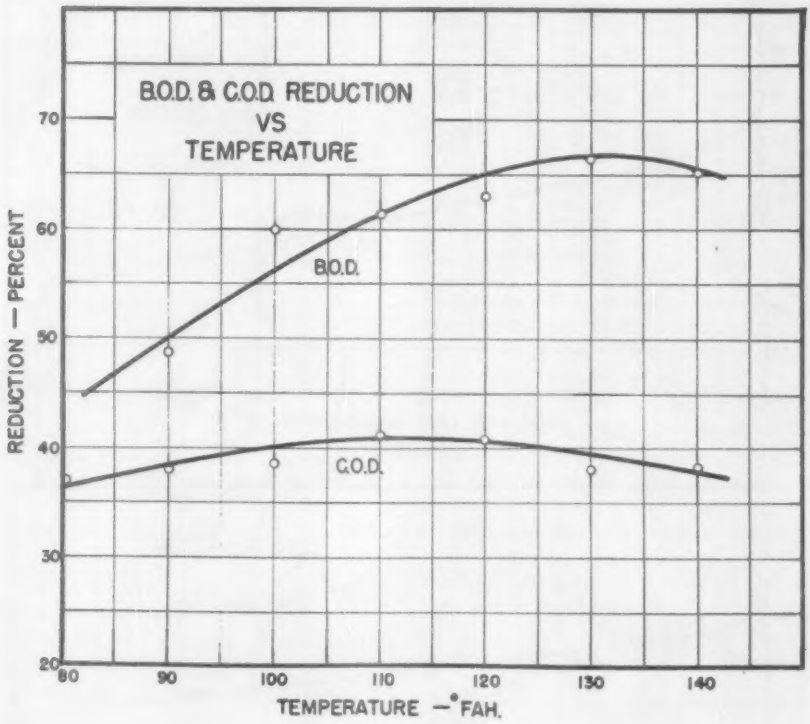

FIGURE 10.

To determine further the effect of temperature, the construction of the previously described filter system was modified by the addition of electric thermo-regulators to the settling tanks to permit adjustment of the recycle temperature. Unit operation was ± 2° F. of the specified temperature throughout the tests.

The synthetic wastes and operating conditions previously described were used to reduce the number of variables. Median pH values were 7.14 for the influent and 7.24 for the effluent, with median B.O.D. and C.O.D. loadings of 107 and 226, respectively.

Operation at a particular temperature was maintained for several days and the values for the average B.O.D. and C.O.D. reductions at each of the ten-degree levels are given in Figure 10. Sixty test runs were made with each filter. In general, little variation was noted in C.O.D. removal between 80 and 140° F. However, B.O.D. removal increased from 44 to 65 per cent between 80° and 120° F., after which it leveled off.

Detergent Effects

The final biofilter pilot-plant investigation was made to study the effect of detergents. The particular detergent used was a sodium salt of an alkyl-aryl sulphonate. Since wastes from the producing plants enter the sewer system, it was desirable to determine their effect on the proposed treatment plant. The deviations are greater for the nutrient-added runs, but the decrease in B.O.D. removal at increasing sulphonate loading is not predictable in a mathematical form.

(These deviations are not greater than the accuracy measurable by the B.O.D. tests.)

The median C.O.D. reduction was 54 per cent on the control filter, while the C.O.D. reduction at varying sulphonate loading is shown in Figure 11 and may be expressed as

$$y = 58.0 - 16.65 \log X$$

in which

y = per cent C.O.D. removed and
X = sulphonate loading, p.p.m.

The deviations from the line of regression show a standard deviation of 4.29. The experimental values are all within the three sigma limits. In fact, all the experimental values, with but one exception, are less than two standard deviations from the line of regression. Therefore, excellent agreement is indicated between the actual and predicted values. The form of the reduction *vs.* loading curve is the normal type expected from biological behavior.

Once again it is obvious that the data scatters for the nutrient-added runs. The following is a statistical comparison of the deviations in the C.O.D. removal by the filters:

Case	Tests	Sulphonate Loading (p.p.m.)	Mean Deviation	Experimental Value of t	Tabular Value of t at 5 Per Cent Level
1	14	0–25	4.6	1.77	2.160
2	15	25–100	8.5	3.69	2.145
3	17	100–400	16.3	3.78	2.120

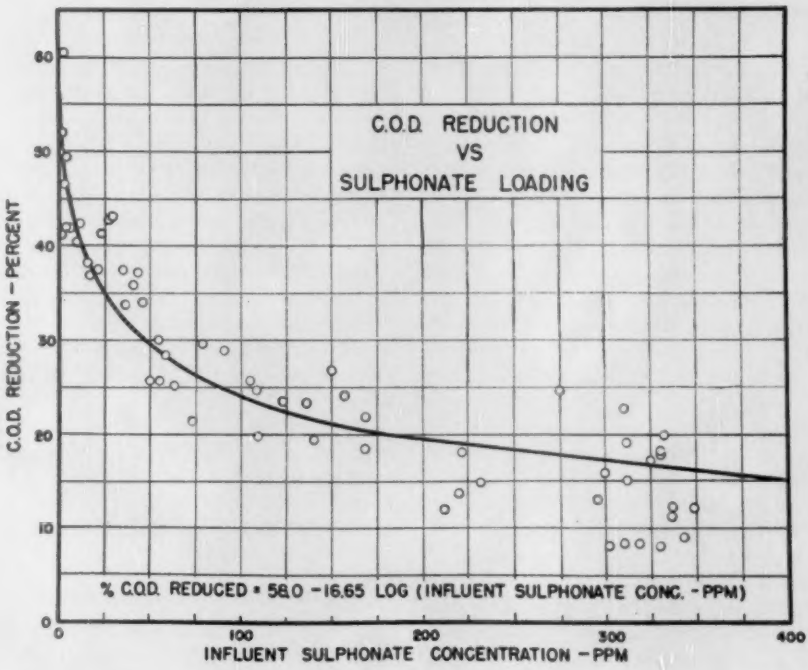

FIGURE 11.

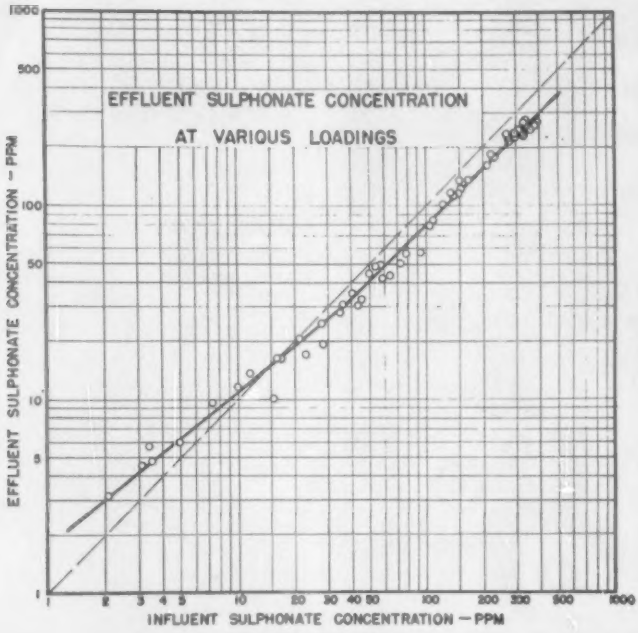

FIGURE 12.

The deviations are insignificant in the 0-25 p.p.m. range which is the most probable in waste treatment plant operation and becomes increasingly significant thereafter as sulphonate loading increases.

An interesting observation was made during this study. At sulphonate loadings less than 20 p.p.m. the effluent sulphonate concentration exceeded the concentration in the influent. However, a constant rate of removal of approximately 23 per cent was noted between 50 and 348 p.p.m. sulphonate (Figure 12). This anomaly may be due to the limitations of the analytical method (i.e. the methylene blue test) or it may be due to behavior inherent to the specific biological system.

Nutrient addition was made in 18 tests to determine if the efficiency of operation could be improved. Ammonium acetate was increased to 63.8 p.p.m. as ammonia nitrogen in 12 runs, and the di-ammonium hydrogen phosphate was increased to 63.2 p.p.m. as phosphorus in six runs with no improvement in the B.O.D., C.O.D. or sulphonate removal. The acetate contributed some organic loading but did not appear to affect the treatment. The scatter of the data during the nutrient runs was not readily correlated with any of the operating parameters.

Despite the high sulphonate loading on the treated filter over a two month period, no detrimental effects were noted on the filter zooglea nor in the amount of material sloughed off the filter.

In conclusion, sulphonate showed little effect on filter operation at concentrations of less than 25 p.p.m. At higher sulphonate concentrations a high C.O.D. loading occurs which is not readily reduced.

II. Activated Sludge Studies

One of the initial activated-sludge studies was made with a small glass unit, consisting of a 430 ml. aeration chamber and slender glass tube set-

tling chamber 28.4 cm. long. Tygon tubing was used and aeration was by means of an air diffuser.

A synthetic sludge developed from domestic sewage, milk and soap was added to the aeration chamber. The floc was light and fluffy with a grey appearance at the start-up. As aeration continued the floc developed a heavier red-brown color and an earthy odor. The initial floc concentration was 2,800 p.p.m.

The feed consisted of a composite refinery waste water. Operating conditions varied within the following limits:

Feed (ml./min.)	Recycle (ml./min.)	Recycle Ratio	Aeration Chamber Detention (min.)	Settling Chamber Detention (min.)	Settling Chamber Overflow Rates (gal./sq.ft./day)
10.2 to 40.2	9.0 to 55.4	0.45 to 1.73:1	10.7 to 42.0	16.8 to 66.1	638 to 2230

At feed rates in excess of 20 ml./per minute the floc was broken up and dispersed in the effluent. However, floc growth was excellent at low feed rates, and especially at high waste loadings.

Loadings and reductions are summarized as follows:

Item	Determination					
	B.O.D.[1]		C.O.D.[2]		Oil[3]	
	Loading (p.p.m.)	Removal (%)	Loading (p.p.m.)	Removal (%)	Loading (p.p.m.)	Removal (%)
Median	55.0	53.8	200	40.5	7.2	80.6
Mean	60.2	55.2	268.5	39.3	9.6	84.4
Range	42.6–88.5	28.2–83.0	155–765	8.4–64.0	4–21	75–99.2

[1] Result of 14 test runs.
[2] Result of 63 test runs.
[3] Result of 20 test runs.

These test runs were made during a 25-day period prior to initiation of a study with a larger aerator. The following recommendations are based on this 25-day study:

1. Control of pH and waste loading in the large pilot plant * should be accurate, because shock loads in the waste may destroy biological activity, unbalance the system, and require prolonged periods of time to rebuild the floc.

2. Excessive flow rates and agitation should be avoided to reduce the floc break up, over-oxidation and loss of floc to the effluent stream.

3. Sludge production must be in sufficient quantity to assure that oil does not accumulate on the floc, resulting in an unbalance of the system.

All of these observations were confirmed in the larger pilot plant. During this period of testing with the larger aerator, a composite waste containing waste caustics, acids, litharge washes, sulphonates, polysulfides and oil was obtained. Discharge of these components is periodic and hence B.O.D.–C.O.D. loading fluctuated widely. The 2,300-gal. pilot-plant was operated from September 21 to November 14, 1950. Initially, the

* Aero-Accelator manufactured by Infilco Inc.

unit was seeded with the addition of 2,500 p.p.m. of activated sludge from a domestic sewage treatment plant.

Early operation was at low flow rates to enable the floc to become acclimated. Sufficient air was added to give approximately 2.0 p.p.m. dissolved oxygen in the effluent. As the test progressed, the thruput was raised to 20 g.p.m. and the air supply rate cut back to produce 0.2 p.p.m. dissolved oxygen in the effluent. The aerator rotor speed was varied as a function of the air supply rate. It was necessary to assure sufficient velocity to complete mixing of influent, floc and air; but not so high that air was hurled from under the hood to disturb the slurry in the settling chamber.

The pH of the influent was adjusted between 7.0 and 7.5 by the addition of caustic or sulfuric acid. The wastes were predominately caustic as received at the unit. Composite samples were collected in a refrigerated sampler and submitted to the laboratory three or four times daily. Additional testing, including the determination of the oxidation-reduction potential, was made in the field.

The oxidation-reduction potential as a means of plant control was not fully developed at the time of this study. Correlation of data in this case was not successful.

The loadings and reductions on the pilot plant may be summarized as follows:

Determination	Tests (no.)	Mean	Median	Range
C.O.D. (p.p.m.)				
infl.	144	262	228	129 to 1662
effl.	141	199	172	84 to 746
filt. effl.	71	118	108	48 to 309
B.O.D. (p.p.m.)				
infl.	165	79	71	35 to 417
effl.	157	39	36	13 to 150
filt. effl.	90	19	16.2	5 to 83.3
C.O.D. reductions (%)				
unfiltered	150	23	26.5	−112 to 77.0
filtered	59	54	52.5	22.7 to 77.6
B.O.D. reduction (%)				
unfiltered	159	45	51.0	−48.4 to 87.7
filtered	94	76	76.5	44.6 to 93.5
Phenol	118	87	91.2	26.7 to 97.7
Oil	82	24	42.0	−280 to 85.7

Since small amounts of solids were continuously lost in the effluent, B.O.D. and C.O.D. data were obtained on the filtered effluent. During the study, several upsets occurred to the aerators with subsequently heavy floc losses. These were attributed to excessive oil contamination, shock loading of chemical wastes and loss of pH control. These were the same difficulties indicated in the smaller glass unit previously described and show the merit of the preliminary studies in pilot-plant work. Of particular concern were the high oil contents. The detrimental effect was not that biological activity was impaired but that when the oil-suspended-solids ratio became high, a physical manifestation, loss by density differences caused heavy floc losses in the effluent. Excellent B.O.D. and phenol reductions continued during intervals of high oil contamination.

Shock loading of contaminants contributing high B.O.D.–C.O.D. necessitated high aeration rates and occasionally the maximum permissible air rate was still unable to contribute an effluent dissolved oxygen content. For normal operation, air consumption is limited by economic considerations.

During the final phase of the study, use was made of water softening sludge ($CaCO_3$) as a weighting agent for the floc, and also as an absorption medium for some of the influent oil. However, the unit was badly unbalanced and the time was too short to ascertain the degree of success afforded by this addition, although some degree of improvement was noted.

In general, the study proved that the wastes were treatable by activated sludge, but modifications to adapt the aeration process to refinery wastes and better preparation and conditioning of the influent are indicated.

In the treatment of refinery wastes by high rate activated sludge processes it has been found that the sludge tended to float in the clarification unit.

For this reason, research should be conducted on the use of dissolved air flotation in place of final sedimentation. Dissolved air flotation depends on the principle that the solubility of air is increased at higher pressures. Decompression of the air-liquid mixture releases the air in a finely divided state. This procedure has been used in lieu of sedimentation equipment (4).

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3. Turnbull, Harry, DeMann, J. G., and Weston, R. F., "Toxicity of Various Refinery Materials to Fresh Water Fish." *Ind. and Eng. Chem.*, **46**, 2, 324 (Feb., 1954).
4. D'Arcy, Nicholas A., Jr., "Dissolved Air Flotation Separates Oil from Waste Water." *Oil and Gas Jour.*, **50**, 319 (1951).

MEETINGS OF INTEREST

1956

Feb. 23	**Symposium on Waste Treatment, National Council for Stream Improvement;** Annual Meeting, Waldorf-Astoria Hotel, New York, N. Y.
Mar. 1–2	**Tenth Industry-Faculty Conference,** Louisiana State University Agricultural and Mechanical College, Baton Rouge 3, La. Frank T. Carroll, Jr., Acting Assistant Director, Engineering Experiment Station, Baton Rouge, La.
Mar. 6–7	**Twenty-first Annual Conference, Illinois Sewage Works Operators.** Hotel Leland, Springfield, Ill. Clarence W. Klassen, State Sanitary Water Board, Springfield, Ill.
April 5–6	**Fifth Southern Municipal and Industrial Waste Conference,** Chapel Hill, N. C. Daniel A. Okun, Dept. of Sanitary Engineering, School of Public Health, University of North Carolina, Chapel Hill, N. C.
April 17–18	**Ninth Municipal and Sanitary Engineering Conference,** University of Florida, Gainesville, Fla. Dr. E. R. Hendrickson, College of Engineering, University of Florida, Gainesville, Fla. Conference theme—Atmospheric Pollution.
May 6–11	**American Water Works Association Diamond Jubilee Conference,** St. Louis, Mo. Harry E. Jordan, Secretary, AWWA, 521 Fifth Ave., New York 17, N. Y.
May 15–17	**Eleventh Purdue Industrial Waste Conference,** Purdue University, Lafayette, Ind. Professor Don E. Bloodgood, School of Civil Engineering, Lafayette, Ind.
June 18–22	**Gordon Research Conference on Stream Sanitation,** Kimball Union Academy, Meriden, N. H. Dr. W. George Parks, Director, Gordon Research Conferences, University of Rhode Island, Kingston, R. I. Applications necessary by April 1; conference limited to 100 persons.

PROCESSING OF PHOSPHORUS FURNACE WASTES*

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The phosphorus and phosphate industry produces some of the most basic chemical products, products that are essential to growth and health. Phosphorus is one of the three major nutritional elements used in fertilizers and proper levels of nitrogen and phosphorus are required to sustain bacterial growth in activated sludge systems and in the B.O.D. test. Complex polyphosphates constitute a major component of modern synthetic detergents and phosphoric acid is used in many applications, from soft drinks to rust-proofing. Organic phosphorus-carrying chemicals, phosphates, phosphites, phosphonates and others, find application as nerve gases, pesticides, plasticizers and lubricating oil additives.

Phosphorus, the common constituent of all these chemicals, is usually produced in the elemental form whenever a source of "pure" phosphorus is required. In its most important form, elemental phosphorus is a waxy yellowish substance with a specific gravity of 1.82 and melts at approximately 112° F. Phosphorus oxidizes rapidly in air, especially in the liquid form where combustion is instantaneous, to produce a phosphorus oxide (P_2O_5) smoke with the liberation of large amounts of heat. The oxide is easily hydrolyzed to phosphoric acid in the presence of moisture.

* Presented at 28th Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Atlantic City, N. J.; Oct. 10–13, 1955.

Phosphorus Manufacture

Elemental phosphorus is produced from phosphate rock in an electric reduction furnace as shown schematically in Figure 1. Phosphate rock is blended with coke, the reducing agent, and silica, which acts as a flux, to form the furnace charge. This charge is delivered to the furnace through vertical tubes which penetrate the furnace roof. In the furnace, electrical energy is used to smelt the charge and liberate reduced phosphorus as a gas. A residual molten slag, mostly calcium silicate, remains in the bottom of the furnace and acts as a conductor to carry the heavy electric current between the electrodes. Also produced in a molten state is ferrophosphorus, a heavy, dense reaction product of phosphorus with the iron contained in the phosphate rock. Both products are tapped from the furnace at periodic intervals.

Furnace gases, at 500° to 800° F., contain elemental phosphorus; silicon tetrafluoride, formed from fluorides in the phosphate rock and silica in the gravel; carbon monoxide, from the oxidation of coke and the carbon electrodes; and various other gases in small amounts, such as phosphorus oxides formed as a result of air leakage into the furnace.

Often the gases are passed through an electrostatic precipitator where much of the dust, carried over in the gas stream, is removed. The dust is mainly coke and phosphate rock fines. After being cleaned in the precipitators, the gases enter the condenser

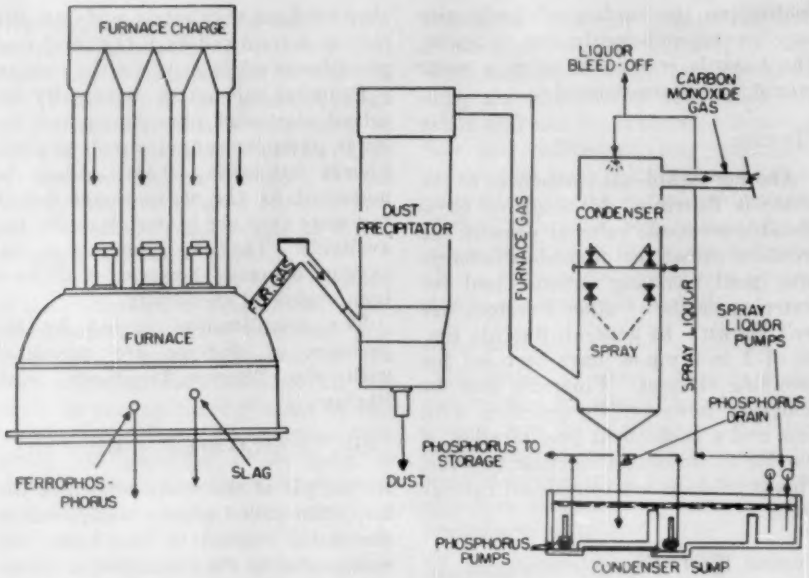

FIGURE 1.—Flow diagram of an electric furnace used to produce elemental phosphorus.

where water sprays cool the gases to 120° to 150° F. The phosphorus is condensed from a gas to a heavy liquid. Silicon tetrafluoride is hydrolyzed to fluosilicic acid (H_2SiF_6). Carbon monoxide passes through the condenser and may either be burned as a waste product or may be used as a source of heat.

The liquid phosphorus and spray water flow from the condenser into concrete sumps. The phosphorus is separated by sedimentation and is subsequently pumped to storage. Condenser spray water is recirculated through the condenser sump and back to the condenser sprays. A proportion of this water is bled off continuously to control the concentration of suspended solids (there is some dust carry-over through the precipitators) and soluble salts (especially fluorides).

Because of the presence of fluosilicic acid and some of the phosphorus acids, the condenser water is usually neutralized with soda ash, ammonia, or some other base to prevent corrosion. If the condenser water is not neutralized, the pH would be 1.5 to 2.0.

Waste Sources and Characteristics

The major source of water-borne waste from a phosphorus reduction furnace is the condenser water bleed-off. Depending on the particular installation, this may vary from 10 to 100 g.p.m. Other waste sources include displacement water from storage tanks or tank cars, water from safety pools under these same tanks, and water used in washing out phosphorus pipe lines.

The most important part of this waste is the elemental phosphorus which is collidally dispersed or is attached to small sludge partices. From an economic standpoint, the value of the phosphorus to be recovered may be considerable. From a waste disposal aspect, the phosphorus is a most objectionable component. As the waste flows through ditches or ponds, the phosphorus and other solids settle out of the water to form small sludge banks and deposits. If the water level varies, these deposits may dry out and the phosphorus then ignites. Sludge

floating on the surface of tanks also may become sufficiently dry to ignite. The hazards created by such a waste cannot be underestimated.

Fluorine

Another significant component of the waste is fluorine. Although of questionable economic value at present, the troubles caused by fluoride discharge into small receiving streams used for watering cattle or other livestock are well known. In general, fluoride limits of 1 to 5 p.p.m. have been set for receiving streams. Fluorides may be removed, however, by reaction with lime and a subsequent precipitation of the highly insoluble calcium fluoride. This problem is not considered further here.

Typical Waste Characteristics

A general characterization of the waste to be treated (if no soda ash or ammonia were added to the condenser water) is:

pH	1.5–2.0
Temperature	120°–150° F.
Elemental phosphorus	400–2,500 p.p.m.
Total susp. solids	1,000–5,000 p.p.m.
Fluorine	500–2,000 p.p.m.
Silica	300–700 p.p.m.
Oxidized phosphorus (as P_2O_5)	600–900 p.p.m.
Reducing substances (I_2)	40–50 p.p.m.

The waste is not particularly strong from a conventional standpoint. The solids loading is not excessive and the oxygen demand is small. The complicating features derive from the presence of elemental phosphorus and fluorine.

For example, the usual, relatively simple, solids analysis is prohibited by the fact that the solids cannot be filtered and dried in air without loss of weight due to phosphorus combustion. The analytical method used takes about 10 hr. and involves distillation and benzene extraction. Water is recovered as the distillate, phosphorus recovered in the benzene (where it is then oxidized with nitric acid and ultimately determined as P_2O_5), and nonphosphorus solids comprise the residue.

Reducing substances, principally dissolved elemental phosphorus and the lower phosphorus oxides such as phosphorus sub-oxide (P_4O), cannot be measured by the conventional B.O.D. test since they are bacteriologically unavailable. They do represent a real oxygen demand, however, and have been evaluated chemically.

The complications caused by the presence of fluorine are associated with the fluorine–fluosilicate equilibrium,

$$SiF_6^{=} + 4H_2O \rightleftarrows 6F^{-} + 4H^{+} + SiO_2$$

As the pH of the waste condenser water, often called phossy water, varies, the solids content of the waste will change due to the formation or disappearance of silica. As the pH increases that is, as the hydrogen ion concentration goes down, the reaction "goes" to the right. Silica is produced and the fluosilicate ion is converted to fluoride ions.

Particle Charge

In addition to the chemical properties of the waste, it is of interest to know something about the physical characteristics, particularly the electrical charge on the surface of the colloidal particles. The character of the charge is especially important when chemical coagulation is being considered. In many coagulation systems, the colloidal particles being agglomerated are negatively charged, and a positively charged particle or ion is used to neutralize the electric potential. With the electrical repelling forces reduced to some suitable value, the individual particles can then be easily agglomerated. Usually aluminum or iron salts such as alum or ferric chloride are used as sources of positively charged counter-ions.

In this waste, however, the particles are positively charged and the com-

mon coagulants are of no avail. Furthermore, the waste as received normally has a temperature of 130° to 140° F., so that the phosphorus present is in a liquid form as a suspension of colloidal droplets. Whether the physical-chemical theories developed for the surface reactions of solid colloidal particles are equally applicable to liquid particles is open to question.

Finally, consideration must be given to the variations in surface characteristics caused by the electrostatic precipitators. Proper operation of the dust collectors requires that all the particles passing through must be ionized by the addition of electrons, thus giving the particles a net negative charge. When the particles are later suspended in water, however, they have a net positive charge. The reason for this is not clear at present. It is well known that these particles, in a dry state, will retain their modified surface characteristics. No data are available, however, on the reaction that takes place when the particles are immersed in water.

Complications arise, however, because the precipitators do not operate continuously. Occasionally, because of dust build-up, these high voltage units ground out and must be cleaned. Particles passing through the precipitators under these conditions do not become similarly charged, and this undoubtedly changes the surface characteristic of the particle. While direct determinations of electrical charge have not been made for particles by-passing a grounded condenser, these particles are known to be more difficult to coagulate and settle out.

Potential Treatment Methods

Treatment of this waste may be approached from either a disposal or a recovery standpoint; both have been attempted.

Lagooning

If waste disposal is the only objective, lagooning seems to be the simplest technique. Fluorine can be removed, if necessary, by the addition of lime. With sufficient storage capacity available, the phosphorus and solids will settle and can be removed. Evaporation and infiltration may materially reduce the waste volume and in some cases eliminate the waste problem completely. Trouble may ultimately be encountered when the lagoon begins to fill up, because the phosphorus bearing sludge is a potential hazard.

Chlorination

Oxidation by chlorination is a second method by which the phosphorus hazard may be eliminated. The oxidized phosphorus is incombustible since it is converted into a water soluble form such as phosphorus or hypophosphorus acid.

Separation

If phosphorus recovery is considered desirable, several separation techniques may be used. The separation conceivably could be effected by plain sedimentation, coagulation and sedimentation, filtration, or centrifuging. All these methods have been attempted, but only coagulation and sedimentation seems completely acceptable for large flows. Laboratory and pilot plant experiments were carried out, therefore, to determine the details of such a process.

Determination of a Coagulation System

In attempting to establish a good coagulation system, the first requirement is a good coagulant. It has been mentioned previously that some of the flocculating agents which, from a sewage or normal waste disposal standpoint, are considered standards, such as alum, ferric sulfate, ferric chloride or lime, were of no value. This is mainly because the suspended particles in phossy water are not predominantly negatively charged.

In recent years a whole series of new coagulating agents, anionic and cati-

onic polyelectrolytes, have been developed. Many of these coagulants were compounded for use in industrial processes such as the settlement of slimes resulting from hydraulic mining operations. Attempts to use these polyelectrolytes were not successful, although better results were achieved than with the "low molecular weight" inorganic compounds tried first.

Protein Coagulants

Since higher molecular weight materials seemed to have some advantages, it was decided to investigate proteins as coagulants. Further, proteins as a class exhibit strong amphoteric properties. That is, on the acid side of the isoelectric point the protein molecule has a positive electrical charge while on the alkaline side it has a negative charge. This occurs because of the presence of "zwitter ions" in the protein, such as $^{+}NH_3CH_2COO^{-}$. In acid media, the positive ion $^{+}NH_3CH_2COOH$ is formed while in alkaline media, the negative ion $NH_2CH_2COO^{-}$ is formed. The degree of ionization, or the voltage of the electrically charged particles, varies depending on the pH of the system compared with the pH at the isoelectric point. Thus, by adjusting the pH, it is possible to obtain a large molecule with either a positive or negative charge and, further, to control the electrical potential, or voltage, so that it just balances that of the particles to be coagulated.

Laboratory Studies

Extensive screening experiments in a laboratory stirring apparatus confirmed the applicability of proteins. The effect of pH on coagulation efficiency was studied for all proteins. In some cases a rather narrow pH range within which the proteins had to operate was found while in other cases the tolerance was large. Table I lists some of the proteins studied, the pH range in which they operate, and the coagulant concentration required.

TABLE I.—Concentration and pH Range of Selected Protein Coagulants

Protein	Conc. (p.p.m.)	pH Range
Blood	20–280	5.5–6.0
Blood albumin	60–200	4.0–4.4
Casein	25– 75	3.0–4.0
Glue	30–200	5.5–6.0
Gelatin	10–150	5.0–5.8
Peanut protein	90–250	2.0–5.0
Soya protein	60–150	3.6–5.6

Control of the pH in a phossy water system is not as straight-forward as might be hoped. In initial pilot unit tests, a pH of 5.8 was selected as being desirable from a coagulation standpoint.

The pH was adjusted by adding soda ash to the condenser sump. After about a week of operation considerable scale was observed in pipe lines and in the pilot unit settling tanks. This scale, formed because of the fluorine–fluosilicate equilibrium which precipitates silica as the pH is increased, accumulated at rate of about 0.01 lb. per square foot per day, or about 1/100 in. per day.

Furthermore, the solids formed were not only depositing as scale but were also acting as weighting agents. As the pH increased, more solids were formed and the settling rate of the coagulated particles increased accordingly (Figure 2). The primary trouble was that the solids were being produced in the condenser sump rather than in the coagulation zone where they were required.

It would be possible, of course, to have two pH control points. The control point in the condenser sump would hold a pH level sufficient to prevent corrosion and the other in the coagulation tank would control the process pH. This obviously would be a costly and cumbersome system.

Rather than re-pipe the experimental equipment and purchase another pH recorder-controller, clay was added with the coagulant as a substi-

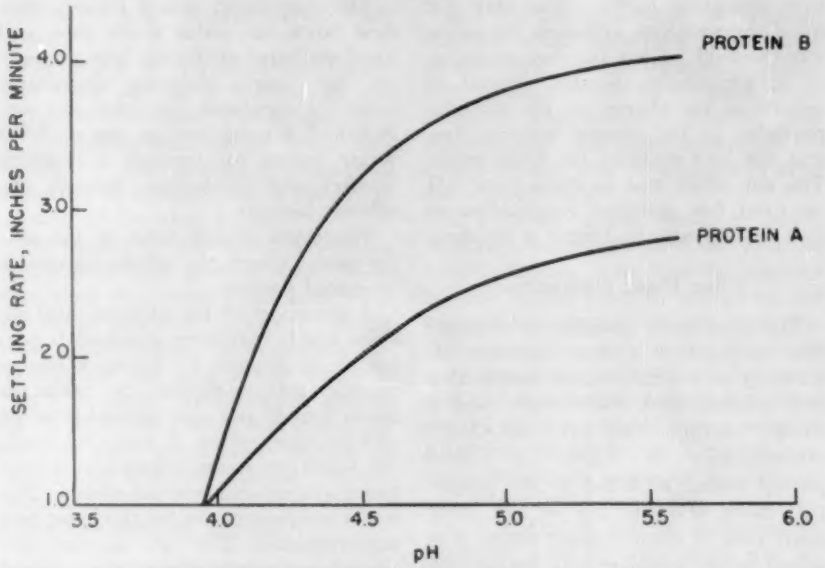

FIGURE 2.—Effect of pH on the settling rate of particles coagulated from raw phossy water by the use of proteins.

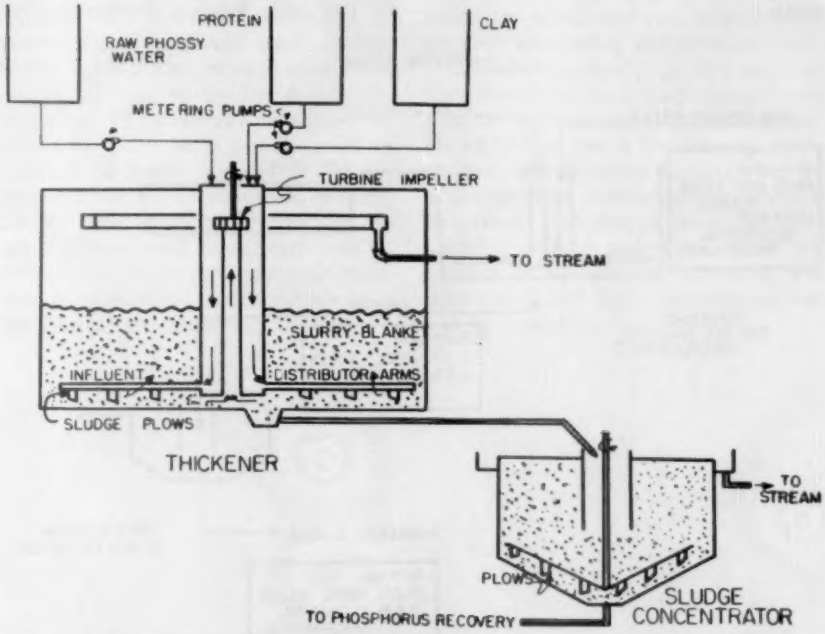

FIGURE 3.—Flow diagram of pilot plant equipment for recovery of phosphorus from phossy water.

tute weighting agent. The clay did solve the problem, although its action was two-fold. First the clay particles, being negatively charged, helped to neutralize the charge on the colloidal particles in the phossy water. Second, the clay replaced the silica solids. The net effect was to reduce the pH required for optimum coagulation so that scaling was no longer a problem.

Pilot Plant Operation

The pilot units installed to evaluate this coagulation system consisted essentially of a combination coagulation and settling tank (thickener) and a sludge concentrating tank as shown schematically in Figure 3. Raw phossy water, adjusted to the proper pH, flows into the top of the outer draft tube of the thickener where it is mixed by the impeller with the protein and clay. Recycle sludge is pumped up the inner draft tube by the impeller and is also mixed with the raw waste.

The coagulated mixed liquors then flow down the outer draft tube and are distributed uniformly into the tank by the slowly rotating distributor arms. Coagulated particles are settled and filtered out as the clarified water passes up through the slurry blanket and discharges through the effluent launder.

Thickened sludge flows to the concentrator where the solids content is increased further.

A summary of the experimental results which have been obtained is presented in Figure 4. Using a sludge recycle rate sufficient to maintain about 5 to 10 per cent settleable solids (by volume, after 5 min. settling), the solids were concentrated to a very heavy phosphorus-bearing sludge. The water temperature in the thickener was approximately 125° F. so that the phosphorus particles were in the liquid rather than solid form. The turbidity of the effluent varied from 40 to 250 p.p.m. depending on the pH, being less at low pH levels. Normally, the

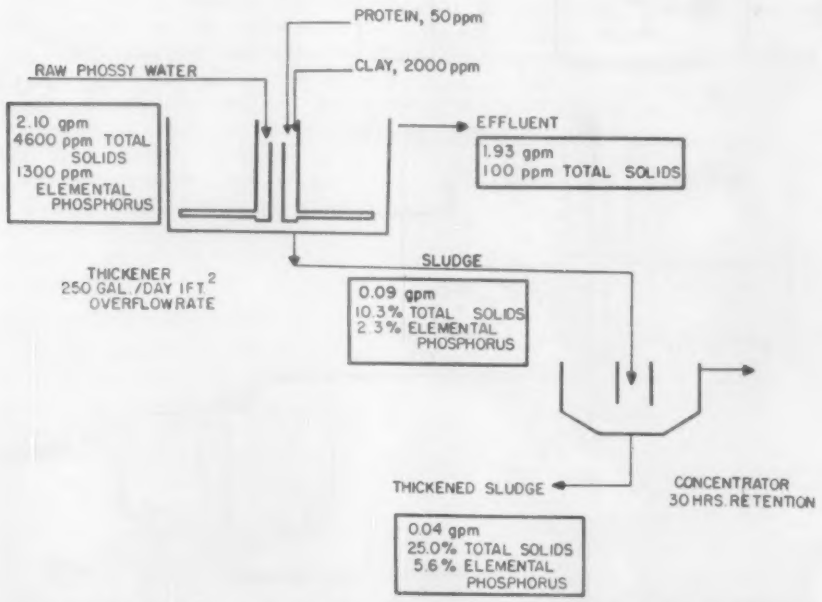

FIGURE 4.—Summary of pilot unit operation materials balance.

turbidity was approximately 100 p.p.m.

The solids in the thickener underflow were increased further in the sludge concentrator. The temperature in this latter unit varied from 100° to 150° F. but sludge concentration seemed to be independent of the physical state of the phosphorus.

The operating pH varied depending on the particular protein used. However, even with waste conditions varying from relatively "clean" phossy water (electrostatic precipitators operating 100 per cent of the time) to "dirty" water (precipitators operating 25 to 50 per cent of the time), the proteins were remarkably capable of coagulating and precipitating the colloidal and suspended particles without a re-adjustment of the pH.

Conclusions

1. Phossy water, a waste product in the production of elemental phosphorus by the electric furnace process, contains from 1,000 to 5,000 p.p.m. of suspended solids. These solids include 400 to 2,500 p.p.m. of elemental phosphorus distributed as liquid colloidal particles. The electrical charge on the suspended particles is usually positive, although it varies depending on the operation of the electrostatic precepitators. The chemical equilibrium between fluoride and fluosilicate ions introduces an important source of variation in suspended solids which is a function of pH changes.

2. Potential treatment or disposal methods which have been investigated or attempted include lagooning, oxidizing, settling (with or without prior chemical coagulation) filtering and centrifuging. Of these techniques, coagulation and settling appears to offer the best solution.

3. Common water works or waste disposal coagulants such as alum or ferric chloride were not satisfactory because of the positive charge on the suspended particles. Newer, inorganic polyelectrolytes such as the Hyamines gave improved performance but were still not satisfactory. High molecular weight protein molecules at a suitable pH level (which varies for each protein) produced excellent coagulation and were highly successful.

4. The proper pH for optimum coagulation with proteins alone was obscured at higher pH levels by the formation of silica which tended to encrust the pipe lines. The addition of clay, as a weighting agent with the coagulant eliminated the scale problem without decreasing the settling rates.

5. Under reasonably proper coagulation conditions in a pilot unit, it was possible to concentrate suspended solids 40-fold so that a 25 per cent solids product was obtained by a simple coagulation and sedimentation process. Dispersed phosphorus droplets were handled under water so that the amount of phosphorus recovered was maximized and little phosphorus was lost by oxidation.

Stream Pollution

SLIME FORMATION IN POLLUTED WATERS *

II. Factors Affecting Slime Growth †

By H. Heukelekian and E. S. Crosby

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Biological slimes exhibit a number of relationships to man's environment, both advantageous and disadvantageous. In marine environment slimes affect the fouling of ship bottoms and the life of wood piling. In streams they play a role in self-purification, and in the formation of secondary sludge deposits. In water pipes their growth may result in the production of tastes and odors, and in tuberculation causing an increase in frictional losses. Condenser and cooling systems often are rendered less efficient by slime growths.

Slimes in sewers are associated with hydrogen sulfide production and with subsequent corrosion of concrete. In activated sludge and trickling filters, they are responsible for the purification of sewage. In industrial processes, such as paper making and citrus fruit processing, their growth often produces an unfavorable effect in the product.

It is not to be expected that the nature of the organisms in all these environments are the same. They have one similarity, however, in that they form growths attached to some submerged surface (concrete, wood, metal, stone or transite) or to some suspended particle (clay, silt or organic matter). In water pipes, the food supply of the slime growths may be the result of autotrophic oxidation of ferrous iron or ammonia. In surfaces exposed to light, food may be produced from the photosynthetic reduction of carbon dioxide. Traces of organic matter provide the necessary food for the slime growth in sea and unpolluted fresh waters, while in polluted streams and in sewage the concentration of organic matter is much higher.

Laboratory experimental conditions do not often readily simulate the conditions in the natural environment. Also, it generally is difficult to study the organisms responsible for slime formation by cultural methods. For these reasons, there is a dearth of information regarding the nature, causative organisms, food requirements and other factors influencing slime formation in these various environments. Zobell and his associates (1)(2)(3) have described many fundamental aspects of attached growths in salt water substrates, and Henrici (4)(5)(6) has studied the occurrence and distribution of slime-forming microorganisms in lakes. The purpose of this paper is to present experimental data obtained from laboratory and field studies on the influence of certain factors on slime formation in sewage.

Relative Weight of Slime in Laboratory Methods

The amounts of slime growth obtained on test slides in the laboratory

* Paper of the Journal Series, New Jersey Agricultural Experiment Station, Rutgers University.

† Part I appeared in This Journal, 27, 12, 1391 (Dec., 1955).

TABLE I.—Comparative Slime Weight Increases in Sewage[1] Determined by Laboratory Methods Over 26-day Period

Days	Static		Accumulative Dry Weights[2] (mg. per slide)					
			Submerged Dipping[3]		Alternate Dipping[3]		Air Lift Recirculation[4]	
	Range	Av.	Range	Av.	Range	Av.	Range	Av.
4	0.4	0.4	0.4–0.5	0.5	2.2–2.7	2.5	3.5–4.4	4.0
6	0.9	0.9	3.1–3.5	3.3	4.7–4.8	4.8	—	—
8	0.4–0.7	0.5	2.8–3.8	3.2	6.4–7.5	6.9	8.8–9.9	9.5
10	1.5–1.9	1.7	4.4–5.1	4.7	12.0–12.1	12.0	13.1	13.1
12	0.6–1.7	1.0	6.3–7.4	6.8	12.4–14.3	13.1	—	—
14	1.6–2.4	2.1	8.7–9.3	9.1	15.3–17.1	16.3	—	—
16	1.9–2.3	2.1	8.4–9.5	8.8	16.5–18.0	17.2	31.3–42.0	35.3
18	3.4–4.9	4.1	10.4–11.7	11.0	23.8–25.8	24.6	—	—
20	2.9–3.9	3.3	11.8–13.2	12.3	21.1–23.7	22.6	29.4–32.9	31.3
22	5.4–6.3	5.8	13.2–16.7	15.4	26.0–27.2	26.5	—	—
24	3.5–3.9	3.7	13.3–19.5	17.4	25.7–26.9	26.3	12.4–20.9	17.7
26	4.5–5.8	5.2	21.0–22.9	22.1	29.6–31.7	30.5	—	—

[1] Screened settled sewage.
[2] Average weight, three slides; slide area, 0.4 sq. dm.
[3] Dipping rate 3 r.p.m.
[4] Recirculation rate 12 l. per hour.

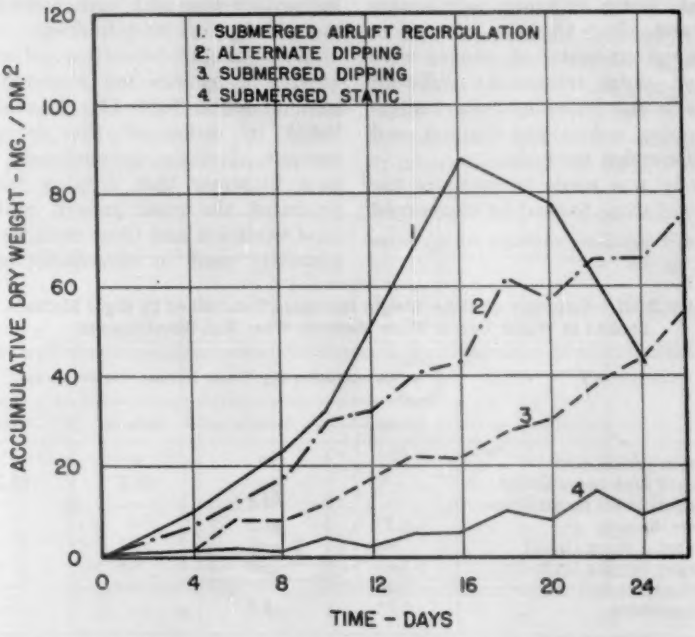

FIGURE 1.—Slime growth using different methods of submergence.

TABLE II.—Comparative Slime Weight Increases in Sewage Determined by Laboratory Methods At 5- and 8-day Periods

Method	Days (No.)	Slides (No.)	Actual Dry Weight (mg. per slide[1])				Calculated Wt. (mg. per sq. dm.) Av.
			Min.	Max.	Range	Av.	
Submerged dipping[2]							
Cylinder[3]	5	10	0.3	1.2	0.9	0.7	1.8
	8	20	1.8	4.2	2.4	2.7	6.8
Tank[4]	5	16	1.1	2.0	0.9	1.5	3.7
	8	32	3.3	6.7	3.4	4.7	11.7
Tank recirculation[4]	5	24	3.0	23.9	20.9	15.0	37.5
	8	32	14.1	36.0	21.9	25.9	64.7

[1] Slide area exposed, 0.4 sq. dm.
[2] Screened, settled sewage changed daily.
[3] 1.5 l. per cylinder, sewage depth 29.5 cm.
[4] 36.0 l. per tank, sewage depth 28.0 cm.

by various methods of submergence in sewage were compared. Slides were sampled every two days from the 4th to 26th day. The weights of slime accumulated are summarized in Table I and are plotted in Figure 1. The weight data show a maximum slime accumulation occurred in the submerged air lift recirculation method, the peak being 88.2 mg. per square decimeter,* after 16 days immersion. Decreasing amounts of slime were found on slides treated by different methods in the following order: alternate dipping, submerged dipping, and static immersion methods.

A study was made to compare the amounts of slime formed by submerged dipping, in cylinders and a tank, with that formed by the tank recirculation (mechanical stirring) method. Slides were sampled after 5 and 8 day immersion periods, for determination of slime weight. Table II shows a maximum slime accumulation occurred in the tank recirculation method; decreased accumulation resulted by submerged dipping in a tank, and by submerged dipping in cylinders.

Slime weight data from all studies (including studies not reported) are summarized in Table III. Methods are listed in order of the decreasing amount of slime accumulated. The data illustrate that dipping methods produced the most growth with the least variation and these methods were generally used in subsequent labora-

* A square decimeter contains .01 sq. meter or 15.50 sq. in.

TABLE III.—Summary of Slime Weight Increases Determined by Eight Methods Studies in Which Two or More Methods Were Run Simultaneously

	Accumulative Dry Weight Increase (mg. per sq. dm.)			
	Series 1	Series 2	Series 3	Series 4
Immersion period, days	7	8	8	7
Submerged tank recirculation	—	—	64.7	65.7†
Submerged air lift recirculation	—	23.7	—	—
Alternate dipping	4.2*	17.2	—	—
Submerged dipping (tank)	—	—	11.7	—
Submerged dipping (cylinder)	—	8.0	6.8	—
Exposed air lift recirculation	3.2*	—	—	—
Static immersion	0.7*	1.2	—	—

* Unchanged settled sewage; all others changed daily.
† Continuous change.

tory studies. Differences in the growth level by any one method, from one series to another, were ascribed to sewage differences.

Comparison of Laboratory and Field Conditions

Before proceeding with detailed studies it was desirable to compare laboratory methods of slime growth with the amounts of slime formed on similar surfaces under field conditions.

Field studies were all carried out at the Plainfield Joint Meeting Sewage Treatment Plant. Weighed slides in copper holders were submerged in the primary clarifier effluent channel (station 4), in the trickling filter effluent well (station 5), in the final clarifier effluent channel (station 6) and in the receiving stream approximately 100 ft. above (station 7) and 100 ft. below (station 8) the point of effluent discharge. Slides were removed at weekly intervals from the 6th to 35th day. The slime weight data are illustrated graphically in Figure 2.

Plant operation at the time of this study involved prechlorination of raw sewage and post-chlorination of the effluent, with a portion of the chlorinated effluent being recirculated to the dosing tanks and trickling filters. No residual chlorine either from pre- or post-chlorination was detected at any of the slide immersion stations.

A comparison of slime accumulation in sewage under laboratory and field conditions shows that the maximum growth rate (65.7 mg. per square decimeter per 7 days) occurred under the laboratory conditions using the submerged tank recirculation method. Higher peaks occurred under field conditions, but only after longer immersion periods. When the amount of accumulated slime was large, there was a greater variation in replicate samples in both field and laboratory studies. Newcombe (7) has reported similar variation with slides submerged in fresh water lakes.

Graphical presentations of laboratory (Figure 1) and field data (Figure 2) show that, by regular sampling, a definite slime growth curve was evident for each condition. In the laboratory study of slime growth, a peak followed by a sloughing loss was more evident than in the field study.

In field studies, the substrate characteristics at the primary clarifier effluent location correspond to those of the screened and settled sewage used in laboratory studies. It is of interest

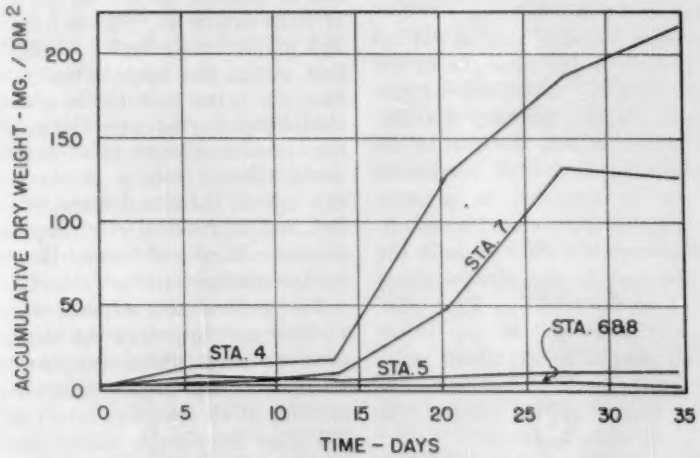

FIGURE 2.—Slime growth in sewage treatment plant.

to note that the submerged dipping laboratory method gave slime growth values closest to the growth obtained on slides in the field test where slides were submerged in the clarifier effluent channel (station 4).

The slime growth in field tests on slides submerged in the trickling filter effluent and final clarifier effluent did not approach the quantities obtained in the settled sewage, while slides immersed in the stream above the outfall gave quantities of growth only slightly less than those obtained in settled sewage. On the other hand, slides submerged in the stream below the outfall gave extremely small amounts of growth comparable to the trickling filter and final effluent values.

One of the important variables in these different locations is the concentration of food material. It is highest in the settled sewage and is greatly reduced by passage through the trickling filter and secondary clarifier; it is further reduced by dilution after discharge to the stream. The field results, however, cannot be entirely explained on the basis of food concentration. The stream above the outfall is certainly lower in food concentration than the effluent from the filters, plant effluent or the stream water below the outfall, and yet a much higher slime growth was obtained at this location.

The second variable that might influence growth is the presence or absence of oxygen. Dissolved oxygen was absent in the primary clarifier effluent, while it was present in all other locations. Anaerobic conditions should not be expected to produce greater growth than aerobic conditions. Furthermore, differences in the results obtained in the stream above the outfall, in the trickling filter effluent, plant effluent and stream below the outfall cannot be explained satisfactorily on the basis of dissolved oxygen alone, since dissolved oxygen was present in all these locations.

The third variable is the chlorination factor. Although there was no residual chlorine detected either from pre-chlorination treatment which might affect results in the primary effluent, or from post-chlorination which might effect the values obtained in the filter effluent, the final effluent and the stream below the outfall, it is conceivable that chlorination affected the results and might explain the observed differences. Post-chlorination, even in the absence of residual chlorine, could retard the accumulation of slime in the filter effluent, final effluent and the stream below the outfall, by reducing the numbers of microorganisms in the substrate and their consequent attachment and growth on submerged surfaces. It was apparent that there were several factors influencing the results, and a more detailed study was made of the influence of certain factors under controlled conditions.

The Nature of the Surface

Effect of the Size of Exposed Surfaces on the Amount of Slime Formed.

Glass microscope slides (3 in. by 1 in.) and larger glass plates (4 in. by 4 in.) were immersed in a vertical position in settled sewage, plant location (station 3), to determine the effect of size of exposed surface on the amount of slime produced. Slimes from slides and plates were sampled weekly. The first study was made when no chlorine was being used at the plant, and the second during pre-chlorination of the incoming sewage, prior to primary clarification. Only a portion of the raw sewage chlorine demand was satisfied, and no residual chlorine was ever present. Result of both of these studies are summarized in Table IV.

The surface area exposed appeared to be significant, since the larger surfaces of glass plates (approximately 1.1 sq. dm.) had greater accumulations of slime at all sampling intervals than did glass microscope slides (approximately 0.15 sq. dm.) under similar

TABLE IV.—Comparison of Accumulative Dry Weights of Slimes Forming on Glass Surfaces of Different Sizes Under Similar Field Conditions[1]

Days Immersed (No.)	Accumulative Dry Weight (mg. per sq. dm.)		
	Glass Slides (0.15 sq. dm.)	Glass Plates (1.1 sq. dm.)	Weight Ratio (Plates/Slides)
Unchlorinated, Settled Sewage			
6	6.4	29.1	4.5:1
13	14.7	279.0	19.0:1
20	13.9	103.2	7.4:1
27	16.4	191.3	11.6:1
35	6.4	156.5	24.5:1
			Mean 13.4:1
Pre-chlorinated, Settled Sewage			
6	8.7	24.0	2.8:1
14	9.4	19.1	2.0:1
21	63.7	115.0	1.8:1
28	99.5	122.1	1.2:1
35	200.5	260.1	1.3:1
			Mean 1.8:1

[1] At station No. 3, primary clarifier effluent channel.

conditions. The differential slime accumulation on glass surfaces of different sizes was evident both in unchlorinated and chlorinated sewage, and at all intervals, but the differences were greater in unchlorinated sewage.

Effect of Type and Roughness of Surface on the Amount of Slime Formed

The effect of type of surface was studied by comparing the amounts of slime formed on test plates of four materials of varying roughness; concrete, Transite, glazed tile and glass. A specific roughness value for the surfaces employed was not assigned. It was readily apparent by observation that concrete was the roughest surface. Differences in the roughness of other test surfaces were less obvious, glazed tile and glass appeared the smoothest, and Transite was intermediate in roughness. All plates were continuously submerged at the same location in the primary clarifier effluent channel for similar periods. Type of surface was considered the only significant variable.

Results of these studies (Figures 3 and 4) indicate that the surface roughness was not a significant factor in the amount of slime formed. The primary film-forming period is the interval in which roughness of the surface would be expected to be most significant. The test results show that during the first week the slope of the weight line, and thus the slime growth rate, was greatest on Transite, followed by that on concrete, glass and tile. Transite was definitely not the roughest surface. This order held for the preliminary period (first week) in both field studies.

Long range growths in the unchlorinated series appeared to be cyclic, reaching peaks followed by sloughing. Growth of slimes during the period of pre-chlorination follows a pattern of gradual increase. Pre-chlorination also appears to retard the growth rate, extending the period of film development from 13 days (in the unchlorinated study) to at least 63 days; the latter period was the termination of the experiment, at which time the growths were still increasing but had not reached the maximum levels reached in the unchlorinated series. It appears that when the rate of growth is retarded by chlorination there is less sloughing. Weekly intervals of sampling in the unchlorinated series, with rapid and pronounced cyclic fluctuations, failed to indicate the actual location and height of the peaks.

The Nature of the Immersion Medium

Effect of Daily Replacement of the Substrate

Laboratory studies were made to determine the weight of slime accumulated on glass slides in unchanged sewage compared with similar surfaces in sewage that was changed daily. The

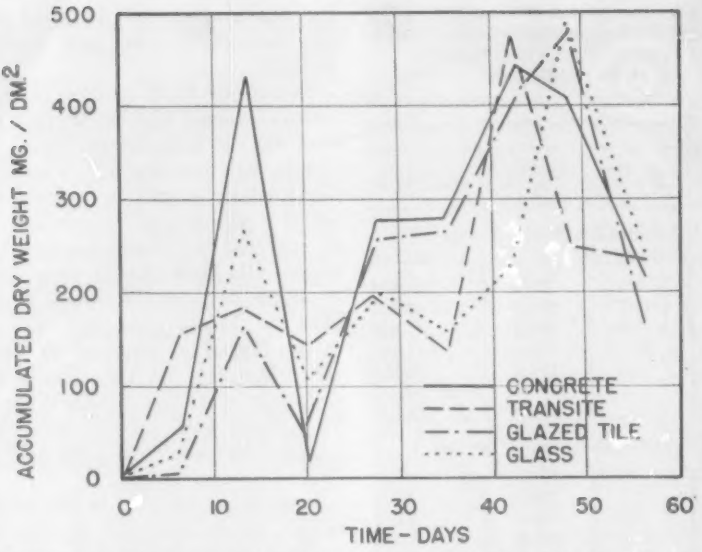

FIGURE 3.—Slime growth in primary effluent channel, unchlorinated sewage.

alternate dipping method was employed, and slides were removed for weighing every 2 to 4 days for a period of 18 days.

The results of these studies (Figure 5) show that rates of slime accumulation were similar for the first four days. After the primary film was formed, weight increases were gradual in unchanged sewage, but rapid in sewage changed daily. This growth pattern indicated either that nutrients became too limiting for rapid growth, or that the septic condition of the unchanged sewage was less favorable for growth.

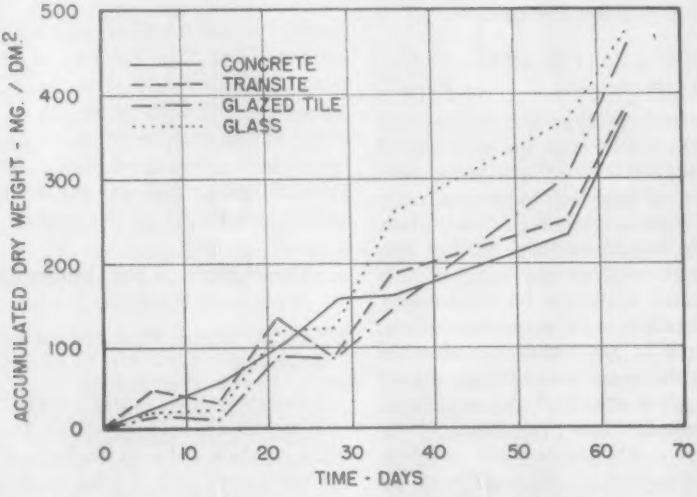

FIGURE 4.—Slime growth in primary effluent channel, pre-chlorinated sewage.

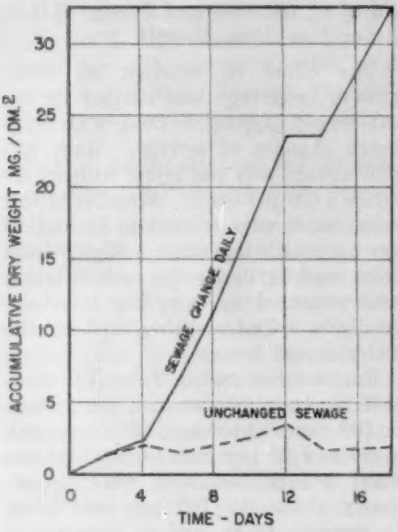

FIGURE 5.—Effect of sewage age on slime growth.

To study the effect of daily batch change of sewage versus continuous flow on slime growth, the tank recirculation method was used. Similar volumes (36 l.) and depths (28 cm.) of sewage were used, and the surface-volume ratios were similar (1.3 slides per liter). For batch change all sewage was siphoned from the tank once every day and the entire volume of fresh sewage was added at one time, the liquid being retained in the system 24 hr.

In the continuous flow procedure, sewage applied in the batch tank studies was pumped to elevated bottles once daily, and was fed to the tanks gradually by automatic siphon feeds. The continuous flow was actually only the gradual addition of a batch of sewage during the following 24 hr. and not analogous to a continuous flow of fresh sewage. The "continuous flow" was actually a portion of the same sewage as that fed to the "batch" tank, except that the former was older and somewhat stale by the time it reached the tank (where it was mixed with sewage still older and even more stale). In the latter procedure, sewage was continuously removed by a siphon outlet. In the continuous flow method there was a gradual accumulation of settleable solids, and an increasing tendency for septicity to develop, both in the feed bottles and in the tank.

As shown in Table V, the data indicate that slime growth was greater with daily batch change than with continuous flow. The previous study (Figure 6) had indicated very significant increases with daily change as compared with unchanged waste, and differences were ascribed either to the depletion of nutrients or to increased septicity in the unchanged waste, or to both factors. In this study, equal volumes of the same sewages were used during the entire experiment, but the continuous flow method delivered staler sewage. The difference in daily versus batch change thus appeared to be due to septicity rather than to depletion of nutrients. Since both tanks contained zero dissolved oxygen at all times, studies were made to evaluate the significance of the degree of septicity.

Effect of Septicity on Slime Growth

Different degrees of septicity, in otherwise similar recirculation tanks, were obtained by providing aeration in one tank, with no aeration other than surface reaeration in the control tank. Settled and screened raw sewage was continuously fed to both tanks, utilizing similar detention periods (12 hr.), volumes (12 l.) and depths (9.5 cm.). As a measure of the degree of septicity, total sulfide con-

TABLE V.—Comparative Slime Growth by Batch and Continuous Flow Methods

Days (No.)	Dry Weight Increase (mg. per sq. dm.)	
	Daily Change	Continuous
5	37.5	0.7
8	64.7	1.7

TABLE VI.—Accumulative Dry Weight of Slime by Aerated and Unaerated Tank Recirculation Method

Days (No.)	Weight Increase[1] (mg. per sq. dm.)		Sulfides (p.p.m.)	
	Unaerated	Aerated	Unaerated	Aerated
5	21.0	43.2	0.6	0.1
6	11.2	32.5	0.6	0.3
7	9.2	12.5	1.7	0.6

[1] Average weight of 8 slides per interval for each condition. Exposed slide area, 0.4 sq. dm. per slide.

tent of the tanks contents was determined.

The results of these studies (Table VI) show that slime growth was greater in the aerated tank than in the control tank. Dissolved oxygen was absent in the control tank after the first day, while in the aerated tank dissolved oxygen was present on two additional days. Total sulfides were present in both tanks, but in considerably higher concentrations in the control tank. Environmental conditions were identical in both tanks with the exception of aeration. Aeration was not vigorous enough to maintain a dissolved oxygen level at all times, but apparently was sufficient to decrease the reducing conditions in the substrate, with an accompanying increase in slime growth.

TABLE VII.—Effect of Sewage Dilution on Slime Weight Increases

Period	Accumulative Dry Weight Increases[1] (mg. per sq. dm.)		
	Sewage Concentration		
	100 per cent	50 per cent	25 per cent
0–7 days	1.7	11.5	8.8
8–14 days	3.4	7.0	13.5
0–14 days	13.1	20.5	22.0

[1] Average weights of 8 slides per interval for each condition.
Area exposed, 0.4 sq. dm. per slide.
Screened, settled sewage.
Submerged dipping method; sewage changed daily.

Effect of Dilution and Sewage B.O.D. Level on Slime Growth

The effect of dilution on slime growth in sewage was studied by the submerged dipping method, with daily batch changes of sewage. Raw, settled sewage was employed without dilution (100 per cent). Aliquots of this same batch were diluted to 50 and 25 per cent with tap water. Eight slides from each cylinder of each dilution were removed at sampling intervals, duplicate cylinders being used for the 100 per cent sewage.

Results presented in Table VII show that the least amount of slime formed in 100 per cent sewage. Slime growth in 50 and 25 per cent sewage did not differ widely, but both were significantly above the 100 per cent level. In general, there was an increase in slime growth with dilution.

The possibility that there existed an optimum dilution or sewage strength led to the reexamination of these data to determine a relationship between B.O.D. and slime growth. A

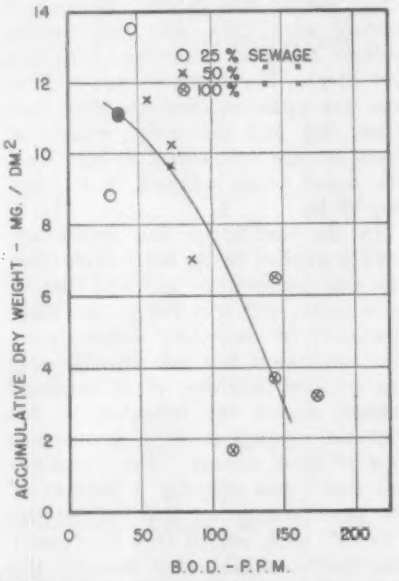

FIGURE 6.—Relationship between sewage B.O.D. and slime growth.

plot of B.O.D. versus slime is given in Figure 6. The curve connects mean values for each sewage dilution, and indicates a definite decrease in slime growth with an increase in B.O.D.

Effect of Added Glucose and Peptone on Slime Accumulation in Sewage

An investigation was made to determine if added carbonaceous or nitrogenous food materials affected the amount of slime formed in domestic sewage. The submerged dipping method was used, since 12 different conditions could be studied simultaneously using the same initial waste. The immersion medium was settled and screened raw sewage. Sewage was changed daily, and nutrients were added to aliquot portions of each batch.

Glucose was employed as the carbonaceous food, and peptone as the nitrogenous material. Concentrations of 5, 10, 25, 50 and 100 p.p.m. were studied for each of these nutrients, and a mixture of 100 p.p.m. of peptone with 100 p.p.m. of glucose was also studied. Slides were removed and weighed after 5, 8 and 12 day periods of immersion.

The data presented in Figure 7 indicate a gradual increase in the amount of the slime formed with an increasing concentration of either glucose or peptone, as well as with a mixture of the two. This trend was uniformly evident for all three time intervals. Glucose produced greater stimulation than a similar concentration of peptone, and the mixture produced an increase practically equivalent to the added effect of glucose and peptone in separate concentrations. The addition of 100 p.p.m. of glucose will add approximately 60 p.p.m. of B.O.D. to the waste. The previous studies have shown that slime growth was inversely related to B.O.D., while these results show that the nature or availability of the oxidizable material is of greater significance than the total concentration, expressed as B.O.D.

Effect of Existing Film on Subsequent Accumulation of Slime

A study was made to determine the effect of the initial film accumulated

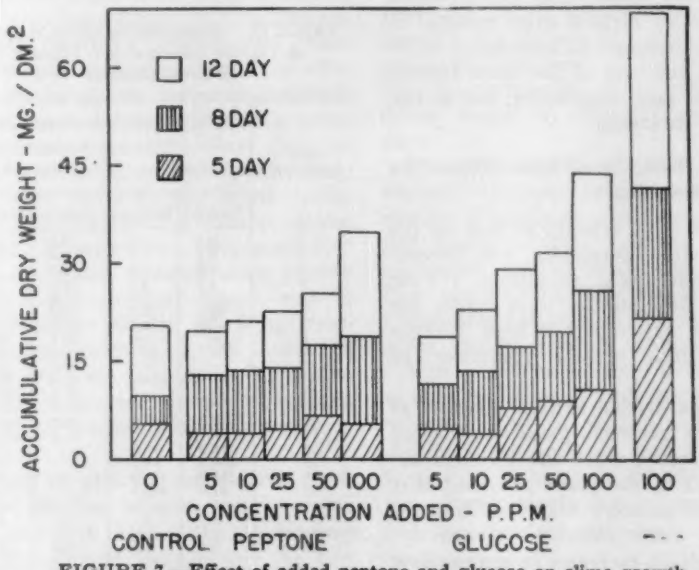

FIGURE 7.—Effect of added peptone and glucose on slime growth.

TABLE VIII.—Effect of Existing Film on Subsequent Slime Accumulation Accumulative Dry Weight Increase[1]

Submergence Period in Days		(mg. per sq. dm.)
A	0–7	1.7
B	8–14	3.5
A+B	0–14	5.2
C	0–14	13.2

[1] Submerged dipping method with daily change of settled and screened sewage. Average weights of 8 slides per interval, 0.4 sq. dm. of area exposed per slide.

on the subsequent amount of slime formed. The submerged dipping method, with daily batch changes of sewage, was employed for the study to permit a maximum number of replicate conditions.

One-half of the initially immersed number of slides were removed, dried and weighed after 7 days. A similar number of new slides were inserted in place of those removed. The new set and the set previously immersed were submerged for 7 additional days. All the remaining slides were then removed, dried and weighed.

Slime weight data presented in Table VIII show that the presence of previously formed slime resulted in a greater subsequent increase of slime than did the sum of the slime formed under the same conditions, but at two separate intervals.

Effect of Velocity on Slime Formation and Sloughing

The effect of velocity of flow on the rate of slime formation was investigated under field conditions. Four separate field studies were made, but only two are reported here because these include greater frequency of sampling.

An open settled sewage channel at the sewage treatment plant was selected for this study, since the physical arrangement of the plant provided that individual primary clarifiers emptied into the same channel successively, with resultant increases in sewage flow and velocity. In the two series reported, slides were immersed at the points of minimum and maximum velocities in the channel. Mean velocities were determined by a Gurley Current Meter at each of the selected locations. Measurements were made at various depths and rates of sewage flow. The channel form was such that velocity at any one station was quite uniform over the flow range (4 to 9 m.g.d.) generally encountered. Mean velocities ranged from 0.40 ft. per second to 1.48 ft. per second. Variations in velocity were not over 0.24 ft. per second. In Study A, slimes were sampled daily for 7 days and weekly thereafter until the sixth week. In Study B, daily sampling was employed for 15 days. Slide weight increases due to slime accumulation, for both studies, are summarized in Table IX.

Results show that up to the 7th day in Series A, and to the 5th day in Series B, the sampling point having the lower velocity exhibited a higher slime accumulation. After these periods, however, the relationship reversed itself, with the location having a higher velocity yielding higher

TABLE IX.—Accumulative Slime Weights at Various Velocities in a Primary Effluent Channel

Days Immersed	Dry Weight (mg. per sq. dm.)			
	Study A		Study B	
	Station 1	Station 4	Station 1	Station 4
1	0.5	1.6	0.9	0.4
2	3.1	1.7	1.3	0.4
3	4.0	2.5	6.5	0.7
4	4.8	4.3	19.0	7.5
5	20.5	5.2	31.7	25.9
6	38.0	13.8	38.8	61.3
7	57.5	35.4	59.8	100.8
8	—	—	60.1	76.7
9	—	—	72.2	81.8
10	—	—	42.9	82.5
11	—	—	42.4	81.9
12	—	—	87.6	124.9
13	—	—	124.0	121.5
14	54.8	143.8	98.2	147.4
15	—	—	81.4	101.0
21	39.2	99.2	—	—
28	208.2	90.2	—	—
35	182.9	105.4	—	—
42	51.6	230.4	—	—
Slides per interval (no.)	8	8	3	3
Mean velocity (ft. per sec.)	0.4	1.48	0.4	1.48

amounts of slime growth. These appear to be valid results, despite the two exceptions shown in later periods of Series A when sampling periods were once in seven days when the peaks might have been missed. The results in Series B, in which a daily sampling schedule was pursued for a period of two weeks, were more consistently higher at the location having a higher rate of flow. It appears that high sewage velocities retard primary film formation, but after a film is established the higher velocities result in greater growth through more rapid replenishment of the food supply to the fixed growth.

Effect of Progressive Sewage Treatment on Slime Formation

Results of a study comparing the slime growth in primary clarifier effluent, trickling filter effluent, final clarifier effluent and the receiving stream above and below the plant outfall have been presented in the general studies to illustrate field growth rates in sewage.

These data (Figure 3) show the maximum slime to be formed in primary clarifier effluent, followed by the amount formed in the stream above the outfall, in the trickling filter effluent, in the stream below the outfall, in the final clarifier effluent, in order of decreasing amounts. Since these results were based on weekly sampling, and were made during plant operation with pre- and post-chlorination, further investigations were needed.

A study was made of slime growth in primary clarifier effluent and in final clarifier effluent, following treatment by trickling filters. Studies were made on microscope slides immersed in horizontal wooden holders, and slide weights are summarized in Table X.

Slime weights for the anaerobic primary clarifier effluent location were identical with those of the aerobic final clarifier effluent location for the first two days and were generally half as great from the second day to the seventh day. Thereafter, slime growth in the anaerobic location was sloughing, as indicated by the weight decrease, with a gradual increase to the 14th day when sloughing again occurred. At the aerobic station, slime generally continued to increase during the entire 15 days, with very minor losses on the 9th and 10th days.

The most significant environmental variable between primary and final clarifier effluents was the dissolved oxygen concentration. Velocity, B.O.D., suspended volatile solids, and seeding from the trickling filter may have been responsible for some minor variations, other factors were of no significance.

Since slime ultimately accumulated to the same level at both low and high velocity flows in settled sewage, as shown in the previous section, the intermediate velocity in the final clarifier effluent would not be responsible for the fourfold difference in slime weight.

TABLE X.—Accumulative Slime Weights in Primary Clarifier Effluent and in Final Clarifier Effluent

Days Immersed	Dry Weight (mg. per sq. dm.)			
	Study A		Study B	
	Station 4*	Station 6†	Station 4*	Station 6†
1	1.6	—	0.4	0.4
2	1.7	—	0.4	0.4
3	2.5	—	0.7	1.9
4	4.3	—	7.5	10.5
5	5.2	—	25.9	46.4
6	13.8	—	61.3	110.5
7	35.4	205.0	100.8	246.0
8	—	—	76.7	299.0
9	—	—	81.8	260.0
10	—	—	82.5	255.0
11	—	—	81.9	304.0
12	—	—	124.9	470.0
13	—	—	121.5	566.0
14	143.8	563.0	147.4	659.0
15	—	—	101.0	669.0
21	99.2	631.0	—	—
28	90.2	860.0	—	—
35	105.4	840.0	—	—
42	230.4	830.0	—	—
Slides per interval (no.)	8	4	3	3

* Station 4 is in primary clarifier effluent.
† Station 6 is in final clarifier effluent.

B.O.D. and suspended volatile solids were both significantly lower in final effluent. They are both related to food supply and, since slime increases rather than decreases were experienced, food was not considered to be a limiting factor. It thus appears that only the increase of D.O., from complete depletion in settled sewage to 10 to 30 per cent saturation in final effluent, was responsible for the 4- to 7-fold increase in the amount of slime present.

Discussion

The factors of slime growth studied fall into two categories: those associated with the nature of submerged surfaces, and those associated with the character of substrate and the surrounding environment. The most important characteristic associated with the nature of submerged surface is the degree of smoothness or roughness. This factor does not seem to exhibit an important relation to the ultimate quantity of slime formed. Smoothness of the surface may retard initial slime formation but, once a coating is established, the initial smoothness of the submerged surface is obliterated and the nature of the biological surface controls further accumulation.

Evidence of a similar nature was obtained from work reported (8) in connection with studies of protective coatings or inhibitors incorporated in concrete. Complete inhibition of growth was not obtained even for short periods with any of 35 coatings, with or without special inhibitors and inhibitors incorporated in concrete itself. It follows that, unless an inhibitor is used which completely stops slime formation, the slime once formed will insulate the action of an inhibiting agent and further accumulation will take place on the slime surface rather than the original surface.

Aerobic Vs. Anaerobic Conditions

In general, it is not the nature of the surfaces, but rather the nature and characteristics of the submerging medium that has an important bearing on slime formation. One of the most important environmental factors was found to be the presence or absence of oxygen in the substrate. Evidence obtained both from laboratory and field studies points to the conclusion that an aerobic environment produces more slime growth than an anaerobic environment.

In laboratory experiments, a lower accumulation of slime was invariably obtained from static submergence, without induced motion of either the substrate or the submerged surface, than was obtained when motion was induced by various means. Under static conditions atmospheric reaeration would be limited and, with deoxygenation brought about by the bacterial utilization of organic matter in sewage, anaerobic conditions would prevail. In the submerged dipping method, where the slides were moved through the liquid, lower accumulations of slime were produced than when the slides were alternately dipped into the sewage and exposed to the air; this may be taken as additional evidence of the effect of a partially aerobic condition. More direct experiments with the effect of septicity also lead to the same conclusion; namely, that a sewage with a greater degree of septicity does not yield as much slime growth.

In aquaria studies, where velocity was imparted by a propeller, atmospheric surface reaeration was not sufficient to maintain aerobic conditions. A greater amount of hydrogen sulfide was produced, with less growth of slime, than in a similarly recirculated tank which was supplied with diffused air in addition to surface reaeration. In aquaria studies, the higher growth of slime with daily batch changes of sewage (as compared with continuous flow of sewage) is also attributed to the greater septicity produced by holding the sewage an additional 24 hr. in the feed reservoir.

The data obtained from field studies, in respect to the amount of slime growth formed on slides submerged in the channel carrying settled sewage in contrast with the slime growth on slides submerged in the final effluent channel after treatment of the sewage, can also be interpreted on the basis of the influence of oxygen. The settled sewage does not contain dissolved oxygen, while the final effluent is well oxygenated.

Slimes grow under anaerobic conditions as well as under aerobic, but the quantity of slime formed is greater where aerobic conditions prevail, such as activated sludge and trickling filter environments in which an adequate supply of air is provided. In sewers, tanks and channels carrying sewage, slime growth attached to the walls will be most pronounced near the air-water interface than at greater depths. Slime formation in overflow weirs is also a common phenomenon.

Influence of Food Supply

The second important factor in slime formation, is the concentration and nature of the organic material. The greater amount of slime growth obtained on slides alternately submerged and exposed *with daily replacement of sewage,* in comparison with the lower growth obtained from a similar procedure *without changing the sewage,* should be interpreted as the influence of food supply. In field studies, the very high slime growths obtained in the stream at a location above the effluent outfall point to the conclusion that high food concentrations are not necessary to support extensive growths. However, the addition of glucose and peptone to sewage results in greater slime accumulation. This is attributed to a change in the character of the food rather than to a concentration effect.

Other Factors

Velocity of flow is the next important factor influencing slime growths. It appears that primary slime formation is more accelerated by low velocities (0.4 ft. per second) than by higher velocities (1.5 ft. per second). Subsequently, however, the higher velocities induce higher growths.

Attention is also called to the cyclic nature of slime accumulation. After the rather slow period of primary film formation, there occurs a rapid accumulation followed by sloughing of the slime growth. The sloughing is not followed by another lag period, but instead by a rapid growth—indicating that the sloughing does not completely remove the slime, but leaves some film to accelerate further accumulation. Experiments to determine the effect of an existing slime film on subsequent increased accumulation of slime substantiate this view. The cyclic nature (accumulation and sloughing) of the slime growth makes it imperative that samples be taken at frequent intervals, lest some of the peaks and troughs of the cycle might be missed. The largest accumulations of slime also produced the widest variations due to sloughing.

Summary and Conclusion

Laboratory and field studies were made to evaluate the influence of certain factors on slime formation in sewage effluents after various degrees of purification, and in stream waters. The results lead to the following conclusions:

1. The nature and degree of roughness of the submerged surface does not affect the ultimate quantity of slime growth except during the initial period of submergence; during that period the smoother surfaces, such as glass and glazed tile, gave somewhat less slime accumulation than did concrete or transite.

2. Aerobic environments produce more slime than anaerobic environments. In the absence of dissolved oxygen, reducing conditions in the substrate are intensified and yield lower slime growths.

3. Both the nature and the concentration of available food affects the slime growth. There is evidence that slime growths increase when the food supply is reduced, either by artificial dilution or by removal and oxidation. The addition of glucose and peptone to the sewage also resulted in greater slime growths.

4. Velocity of sewage flow influences the quantity of slime. During the primary slime formation period lower velocities (0.4 ft. per second) gave greater amounts of growth than higher velocities. After the initial period, higher velocities induced greater growths.

5. Slime growths are cyclic. There is an initial lag period followed by a rapid accumulation, and then by a sudden decrease attributed to the sloughing of the attached growths. This phase is not followed by another lag period, but instead by another period of rapid slime accumulation and then by sloughing of the growths.

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AN INVESTIGATION OF THE TREATMENT OF CABIN CRUISER WASTES *

By William T. Ingram

Adjunct Professor, College of Engineering, New York University, New York, N. Y.

A paper (1) by Ingram and Diachishin presents the essential details of the problems created when cabin cruisers, at anchor in sheltered waters, serve as a base of activities during summer cruises and weekend outings. Water pollution in the vicinity of cruiser anchorage is associated with activity aboard the craft, particular types of activity, boat concentration about a given sampling point, and the physical and tidal characteristics of the receiving water body. The physical evidence of freshly discharged bodily wastes in sheltered waters floating a temporary population is an adequate warning of the public health hazards and suggests strongly the need for treatment of cabin cruiser waste discharges.

Problem of Treating Waste Discharges

Except for the more palatial craft, cruisers have little room for placement of equipment. Any treatment requirement involving extensive cubage or weight is almost prohibitive regardless of its effectiveness. Types of wastes to be disposed include: (a) the bodily wastes; (b) organic refuse developing from food preparation; and (c) miscellaneous refuse including paper, cans, bottles and rags. These wastes become important in the order named. Bodily wastes, though small in volume, offer the greatest public health hazard and the biggest problem in handling.

Galley refuse wastes can be held in containers and later carried to shore along with the miscellaneous refuse. However, any attempt to hold bodily wastes in containers pending later transfer to shore facilities would be objectionable to the cruiser occupants even if the craft were provided with storage tanks of sufficient capacity. A more satisfactory handling of both galley refuse and bodily wastes would consist of a method whereby the cruiser occupants would discharge these wastes to water after acceptable treatment. Criteria for such treatment include:

1. Destruction of waste identity.
2. Elimination of floating solids.
3. Minimization of hazard to bathers.
4. Minimization of hazard to shellfish beds.
5. Minimization of space and weight requirements for treatment equipment.
6. Minimization of operation and maintenance requirements of treatment equipment.
7. Simplification of treatment controls so that craft occupants will, with a minimum of effort, place the treatment processes in operation whenever required.

The general objective of the present study was that of determining what method of treatment of small pleasure craft wastes might be practical and feasible in minimizing the pollution of protected harbors. Investigations were directed to a study of chlorination and maceration of wastes and an evaluation of the bactericidal efficiency of the treatment.

* Presented at 1955 Joint Meeting, New York Sewage and Industrial Wastes Assn. and New England Sewage and Industrial Wastes Assn.; Albany, N. Y.; June 13–14, 1955.

Laboratory Procedures

Standard methods were followed as closely as possible. The procedures used are described below.

To determine the M.P.N. of coliforms in feces the following method was used.

A weighed specimen was macerated thoroughly in a laboratory mixer with 400-500 ml. of water. The sludge was then added to additional water and made up to a volume of 5 l. The volume of flushing water on cabin cruisers varies widely. Hence, 5 l. was chosen as a volume representing minimal flushing and offering a heavy concentration of bodily wastes requiring treatment.

Samples of thoroughly mixed liquor were then transferred by the standard series dilution technique to three lactose broth culture tubes for each of six dilutions from 10^{-4} through 10^{-9}. Gas formation was considered a positive indication of the presence of coliforms.

On some samples plate counts (10^{-3} through 10^{-9}) on agar were also made. Plate counts suggested strongly the presence of inhibitory substances in the lower dilutions and in reporting plate counts only the higher dilution counts were used (10^{-5} and higher).

The procedure used for screening tests on chlorine dosage was as follows. Samples were macerated and mixed as for M.P.N. determination. Portions containing 500 ml. were transferred to beakers containing the required amount of chlorine and thoroughly mixed. After 15 min. aliquot portions were removed to bottles containing sodium thiosulfate and shaken well. The standard series dilution technique (0.1, 0.01 and 0.001 ml. portions in lactose broth) was then followed. Tubes were read at 24 and 48 hrs. Presence of gas was considered a positive indication of the presence of coliforms.

A pilot plant for treatment of wastes was set up in the laboratory. The plant comprised a standard household food wastes grinder, a holding tank with overflow set at 20 l. capacity and a discharge and sampling line. Procedure used in the pilot plant study included the following sequence.

A single test required six specimens of bodily waste including feces, urine, and paper. Each specimen was collected in a waterproof bag and stored at $-20°$ F. until required. Six specimens were thawed out at room temperature until completely unfrozen. Each specimen was weighed to the nearest 0.1 g. The specimen together with 5 l. of sea water made up one charge. A volume of chlorine water made up to give the required dosage in 5 l. of water was then added with the charge to the grinder and discharged to the holding tank. Each succeeding charge was added at 10 min. intervals. The tank held the mixture until admittance of the fifth charge. Five liters of ground waste mixture overflowed after 40 min. and again after 50 min. During the holding period the tank mixture was stirred or rocked gently. A bottom opening wooden baffle in the tank prevented direct overflow of floating foam. Samples were taken on the outlet side of the baffle after 30 min. detention in the holding tank and from the discharge pipe after 40 and 50 min.

One portion of the 50 min. sample was dechlorinated with sodium thiosulfate; another 250 ml. portion was macerated with a laboratory mixer and then dechlorinated and 1 l. was placed in an Imhoff Cone. Series dilutions for coliform determination were carried out according to the standard methods previously described. Imhoff Cone readings were taken at 30 and 60 min. and held for several days for observation. The starch-iodide method was used to determine chlorine residuals.

Results of Investigation

The first series of investigations were intended to determine roughly

TABLE I.—Laboratory Determination of Coliform M.P.N. in Feces

Test	Org./ml.	M.P.N. (10^6)	Sample		
			Feces (g.)	Vol. (l.)	Org./g. of Feces (10^7)
A	$.9\times10^5$	9	77	5	0.584
B	140×10^5	1,400	43	5	162.791
C	4.5×10^4	4.5	44	5	0.511
D	7.5×10^4	7.5	36	5	1.042
E	20×10^4	20	36	5	2.778
F	110×10^4	110	36.4	5	15.110
Average		258	45.4	5	30.5
Excluding B		30			4.0

levels of coliform density to be used as a measure of the magnitude of coliform density to be removed by treatment. A few plate counts were made also to gain some idea of the bacterial population that might grow on agar at 37° C. Several textbooks and references were reviewed, and queries were directed to others of the staff in an effort to find information that could be used as a check on results obtained. Prescott, Winslow and McCrady (2) have indicated that coliform density may vary from 5 million to 500 million per gram of feces.

Six samples of feces were examined for coliform density and the results are summarized in Table I.

Specimens ranged in weight from 36 to 77 gr. and the average of six specimens was 45.4 gr. M.P.N.'s were extremely variable, ranging from 9×10^4 to 14×10^6 per ml. An approximate average M.P.N. based on selection and interpretation of significant numbers was $2{,}580 \times 10^3$ per ml. Excluding one unusually high result the M.P.N. would be 310×10^3 per ml. Translating these data to organisms per gram of feces is not mathematically accurate, but will serve to indicate magnitude of coliform density. A range of 5.8×10^6 to $1{,}630 \times 10^6$ was found. The average of six specimens was 305×10^6 and by excluding the high sample the average was 40×10^6 coliform per gram of feces.

Table II summarizes the data obtained from plate count determinations. The range of samples was 2×10^8 to 192×10^{12} with an average of 62×10^{12} organisms growing at 37° C. per gram of feces. If the low sample count (obviously inhibited growth) is excluded, the average of three samples would be 83×10^{12} organisms per gram of feces.

Table III indicates the results of screening tests intended to show the effect of chlorination on coliform density of macerated feces in water.

Three series of tests using chlorine

TABLE II.—Laboratory Determination of Fecal Organisms Growing on Nutrient Agar

Test	Plate Count		Sample		
	Org./ml.	(10^9)	Feces (g.)	Vol. (l.)	Org./g. of Feces (10^{13})
A	3×10^5	0.003	77	5	.0002
B	$2{,}814\times10^7$	28.14	43	5	3.2721
C	476×10^9	476.0	44	5	54.0909
D	$1{,}383\times10^9$	1,383.0	36	5	192.0833
Average		472.0	50	5	62.4
Excluding A		629.0			83.2

TABLE III.—Effect of Chlorination on M.P.N. of Coliform Organisms in Macerated Feces in Water

Dosage (p.p.m.)	Series					
	D[1]		E[2]		F[3]	
	Chlorine (mg./g.)	M.P.N.	Chlorine (mg./g.)	M.P.N.	Chlorine (mg./g.)	M.P.N.
5	.69	1400+				
10	1.39	1400+				
15	2.08	1400+				
20	2.77	1100+	2.77	1400+		
30	4.16	1100+	4.16	1400+		
40	5.54	200+	5.54	1400+		
50			6.92	1100	6.85	250
60			8.31	95	8.22	250
70					9.59	95
80					10.96	<30
90					12.33	<30

[1] Vol. 5 l., wt. 36 g.; each sample.
[2] Vol. 5 l., wt. 36 g.; each sample.
[3] Vol. 5 l., wt. 36.4 g.; each sample.

dosages ranging from 5 p.p.m. to 90 p.p.m. showed that practically all coliforms were killed at dosage above 11 mg. of chlorine per gram of feces when held for 15 min.

A summary of averaged results (Table IV) obtained when chlorinating and grinding, and holding bodily wastes for a period of time (minimum 10 min.; maximum 50 min.) suggests that a nominal chlorine dosage of 200 p.p.m. is necessary to insure reasonable disinfection. There were some differences between the ground waste discharge and the same waste subjected to additional maceration, but in only one test run at a nominal chlorine dosage of 200 p.p.m. was the macerated waste of lower M.P.N. density than the ground waste. It was reasoned that secondary maceration would expose additional organisms to chlorination if the original grinding were inefficient. For reasons unknown two tests with an average chlorine content of more than 13 mg. per gram of feces gave a higher coliform density in the discharge than did three others at 7 to 8 mg. of chlorine per gram of feces.

Further analysis of chlorine dosages with 200 p.p.m. chlorine are presented in Table V. The average weight of discharge specimens including feces, urine and paper was 154.1 g. with average variations from 125.6 to 217.5 g. Among 30 individual specimens in the 200 p.p.m. available chlorine series the least weighed 22 g. and the greatest weighed 463 g. Average chlorine application varied from 7.01 to 13.41 mg. per gram of specimen with applications to individual

TABLE IV.—Summary of Laboratory Pilot Plant Results on the Chlorination, Grinding and Maceration of Cruiser Wastes

Test	p.p.m.	Chlorine Dosage (mg./g. sample)			Discharge[1]	
		Min.	Aver.	Max.	M.P.N.	Macerated Discharge
1	100	2.56	3.69	5.66	1400+	9.5
2	140	2.48	4.94	6.93	450	110
3	180	3.26	7.53	19.56	250	450
4	200	3.86	8.71	17.30	110	110
5	200	5.37	13.41	44.84	1400+	1400+
6	200	4.15	13.32	23.78	1100	25
7	200	2.16	8.24	17.01	9.5	45
8	200	2.80	7.01	11.71	9.5	250

[1] There was no detectable chlorine residual in any of the discharge samples.

TABLE V.—Waste Treatment with 200 p.p.m. Chlorine Water and Grinding in Standard Food Waste Grinder

Test	Samples (No.)	Average Weight (g./sample)	Chlorine Dosage			M.P.N. Effluent from Holding Tank				Coliform Density (Org./g. charge)	
						Overflow Time (Min.)					
			mg./l.	mg./kg.	mg./sample	−10	0	+10	Mac. +10	+10	Mac.
4	6	140.5	200	189	8.71	1100	110	110	110	783	783
5	6	125.6	200	189	13.41	1400+	150	1400+	1400+	11,146	11,146
6	6	104.4	200	190	13.32	1100	1100	1100	25	10,536	240
7	6	217.5	200	186	8.24	1100	250	9.5	45	44	207
8	6	182.3	200	187	7.01	110	15	9.5	25°	52	1,371
Av. (5)		154.1	200	188	10.1	960	320	530	37°	4,500	2,700
Total	30	770.3				728	147	109	134		
Log. Av.						852	369	307	108		
Av. excl. Test No. 5						618	146	57.5	74.6		
Log. Av. excl. Test No. 5											

specimens ranging from 2.1 to 44.8 mg. per gram of specimen.

Coliform density for the 200 p.p.m. available chlorine series is somewhat erratic. However, the average M.P.N. for the five tests shows a reduction with increased chlorination time and the overflow discharge M.P.N.'s are in the range of reasonable reductions for concentrated sewage. Calculated by the arithmetic average method the discharges at zero time and + 10 min. (sixth charge to holding tank) contain an M.P.N. of 320 and 530 coliform per milliliter respectively, while the latter sample macerated showed an M.P.N. of 370. Calculated by the log average method these M.P.N.'s are 147 and 109 for 0 and + 10 min. discharges and 134 for the macerated sample. If test No. 5, which showed an extremely high M.P.N. at all times except the zero discharge were excluded, the M.P.N.'s would be 146, 57 and 75 respectively by logarithmic average.

Table VI offers information on settling characteristics and particle size. It will be noted that the sewage created in these experiments is quite concentrated with a charge of 2 to 4 per cent of the total weight. Settleable material settled rapidly leaving a brown colloid-like suspension in the supernatant. From 20 to 40 ml. per liter settled in 30 min. and from 21 to 44 ml. per liter settled in 60 min. It is of particular interest to note that even after one week of standing at room temperature no gas formation could be observed and no floating material developed. The supernatant liquor remained turbid. Apparent maximum particle size after grinding was 5 mm. The number of such particles was limited. However, most of the material was less than 1 mm. in diameter.

Practicability of Treatment

On the basis of these laboratory studies it seems efficient to treat

TABLE VI.—Settling Characteristics and Particle Size, Ground and Chlorinated Wastes

Test No.	Av. Wt. (g.)	Av. Cl. (mg./g.)	Per Cent Waste by Weight	Imhoff Cone (ml./l.)		Max. Particle Size (mm.)
				30 min.	60 min.	
1	145.5	3.69	2.7	20	21	1
2	156.3	4.94	2.9	38	41	5
3	175.9	7.53	3.3	40	42	5
4	140.5	8.71	2.7	40	44	5
5	125.6	13.41	2.4	20	21	5
6	104.4	13.32	2.0	21	23	3
7	217.5	8.24	4.0	25	26	5
8	182.3	7.01	3.4	22	24	3
Av.	156			28.2	30.2	

ground bodily wastes with chlorine at rates of 8 to 14 mg. per gram of waste. Provision must be made for a holding tank of suitable capacity to provide for the mixing of the tank contents and a chlorine contact time having a theoretical maximum retention time of 25 min. and a minimum retention time of 10 min.

Effluent diluted with the receiving water on which cabin cruisers will float should be non-detectable in a short period of time following discharge, since the material either settles rapidly or is finely divided and easily dispersed.

It is unlikely that a proportioning device would be installed to weigh and dose individual charges from the toilet. Hence the weight ratio is useful only for estimating an approximate volumetric chlorine dosage. It may be assumed from these study data that a dosage of 1,500 mg. of chlorine per charge will secure an adequate kill of organisms in ground wastes with 10 to 25 min. detention time. Thus, if the volume of water flushing is known or can be regulated the nominal chlorine dosage in parts per million can be established. A holding tank capacity of 10 times this flushing volume should provide adequate holding time for most, if not all possible variations in toilet usage. Even if the flushing water volume was comparable to residential usage, the tank volume should not exceed 25 gal. Such a volume does not appear unreasonable in cubage or weight requirements on a cabin cruiser.

A more serious problem occurs in the determination of a grinding unit. The food waste disposal unit used in the laboratory is heavy and would not fit too well into a drainage system leading from the toilet. It is more massive than need be for the character of wastes to be ground.

Ideally the grinding unit for bodily wastes should be compact, light-weight and capable of shredding paper, rags, sanitary napkins, and bodily wastes to less than 5 mm. in size. It should fit into a closed system operating under pressure if need be. The unit should be powered suitably to operate on the electrical system of the cruiser. In addition, the unit should be so controlled that it will run each time the toilet is flushed.

The complete unit for treating bodily wastes should include a stool and provide for:

1. Water flushing with controlled volumes of water.
2. Adding chlorine in controlled quantity dependent on the water volume, but equal to not less than 1,500 mg. per charge.
3. Grinding all the discharge to a size less than 5 mm. and preferably less than 3 mm.

4. Holding about 10 flushing volumes of ground and chlorinated wastes for 40 to 60 min. to provide an excess chlorine contact time as a safety factor.

5. Controlling the discharge volume from the holding tank equal to the charge volume from the toilet.

6. Draining, flushing, and cleaning the entire system periodically.

Conclusion

These investigations suggest that it is feasible and practical to develop a process for treating bodily wastes in either fresh or sea water by means of grinding, chlorination and chlorine contact detention. Investigations conducted on a field pilot plant scale and incorporating these features merit further consideration.

Summary

Previous studies have shown that untreated waste discharges from cabin cruisers at anchor in protected harbors can create a hazard to bathers swimming in those waters, and to shellfish beds lying under those waters.

A method of treatment of bodily wastes has been studied in the laboratory and the effectiveness of that method in reducing coliform density of toilet charges has been evaluated.

Studies of coliform density indicate a range of 5.8×10^6 to $1,630 \times 10^6$ organisms per gram of feces. After grinding, chlorination and detention, coliform densities can be reduced effectively. With average chlorine dosages of 10 mg. per gram of toilet charge, the coliform density is reduced to an over-all approximation of 4,500 organisms per gram of feces with grinding only and to 2,700 organisms per gram with subsequent maceration. In terms of M.P.N. in the volumes of waste used, the log average of the final discharge water was 109 M.P.N. per milliliter. Average weight of all toilet charges was 154 gr. per charge. Solids settled readily in Imhoff cone tests and averaged 28 ml. in 30 min. No activity was observed in samples held in the cone for a week at room temperature and the settled sludge remained settled. Maximum particle size after grinding was approximately 5 mm.

A chlorine dosage of about 1,500 mg. per charge of material appears to give effective reduction of coliforms when successive charges of ground wastes are held in contact with chlorine over a period of 20 min.

Various parts of a unit for treating wastes aboard cabin cruisers are suggested.

In the laboratory it was demonstrated that bodily wastes could be treated by grinding and chlorination to reduce coliform density and to obtain solids having good settling characteristics. Such wastes, discharged to a receiving body of water large enough to float a cabin cruiser, should disperse and be disposed of without serious hazard to health or creation of aesthetic nuisances.

Acknowledgment

The work described herein was performed at New York University, College of Engineering, under sponsorship of the Interstate Sanitation Commission. The support and counsel of Mr. Seth G. Hess, Chief Engineer, and Mr. Alex Diachishin, Principal Engineer of the Commission, were essential to the conduct of this research. Their assistance is appreciated.

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THE OPERATOR'S CORNER

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AIR FLOTATION STUDIES OF SANITARY SEWAGE*

BY THOMAS T. HAY

Superintendent, Sewage Disposal Plant, Racine, Wis.

The flotation of solids has been practiced for over a century. An excellent review of the development of flotation by G. A. Rohlich (1) develops the history of flotation from the early separation of mineral ores to the use of finely divided air bubbles for the flotation of solids.

Racine Sewage Treatment Plant

The Racine Sewage Treatment Plant is a primary treatment plant with separate sludge digestion and chlorination, serving a highly industrialized community of 75,000. The flow to the treatment plant is variable in quantity and quality and this variation corresponds to conditions usually found in municipal sewage treatment plants.

The sewers in Racine were built over a period of 70 yr. During the early years of sewer construction the size and grade of the lines were not as carefully chosen as they are today. The materials and workmanship also were not of the same quality as now, nor was the work as carefully supervised. The natural terrain of the area was of assistance in maintaining good flow conditions. The first sewers were built for combined flow, while in later years there has been an extensive program of storm sewer construction. However, a large area served by storm sewers still has roof water discharging to the sanitary sewers.

The major industries in Racine are engaged in various metal finishing processes. Accordingly, the characteristics of the sewage are not subject to any unusual seasonal variations. The daily flow has increased since the plant was put in operation in 1938 and twice the average daily flow for a month has exceeded double the designed capacity of the treatment plant. The variation in day and night flow follows a normal pattern ranging from 72 per cent of average to a maximum of 130 per cent. The strength of the sewage has a variation of 3 to 1 between the day and night flows.

The treatment facilities at the Racine sewage treatment plant consist of the following:

1. Pre-chlorination
2. Commutation
3. Grit removal
4. Clarification
5. Post-chlorination
6. Sludge digestion
7. Sludge drying and grinding

Air Flotation Pilot Plant

Expansion of the clarification facilities at the Racine sewage treatment plant is contemplated. Conversion of an existing clarification basin to flotation seems to offer an economical method for increasing the plant capacity. To evaluate the flotation proc-

* Presented at 1955 Annual Meeting, Central States Sewage and Industrial Wastes Assn.; Rochester, Minn.; June 22–24, 1955.

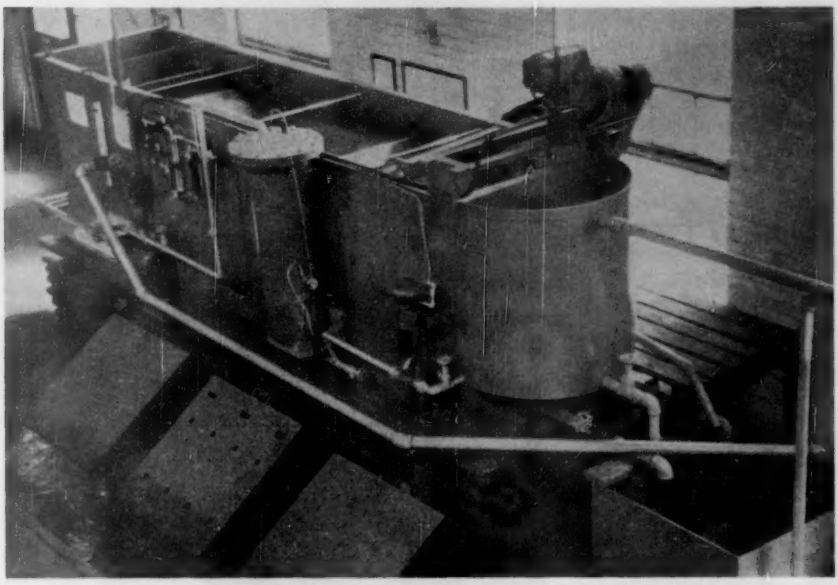

FIGURE 1.—The air flotation pilot plant. Note the slotted influent baffles.

ess a "Float-Treat" † pilot plant (Figure 1) was studied as a means for primary treatment of the Racine municipal waste.

The flow to the pilot plant was obtained from the channel which carries the grit chamber effluent to the primary clarifiers. The flow was pumped to a weir box that had fixed side weirs to maintain a constant level. An adjustable 90° V-notch weir was provided to meter the flow to the flotation unit.

Figure 2 shows the general flow diagram of the flotation process. The influent enters the mixing zone just before the recycle distribution header, where it is mixed with the air-charged recirculated effluent (recycle). The flow then passes through a slotted baffle into the flotation tank where the solids are floated and skimmed from the surface. The clarified liquid is withdrawn from the lower portion of the tank. A portion of this clarified effluent is recirculated under pressure by a pump and air is introduced. The recirculated flow is measured with an orifice plate and a manometer and the air is measured by a "Flowrater" and introduced into the discharge line from the pump. The air-water mixture is held in a pressure tank for a short period of time to permit the air to dissolve in the water. This solution is then discharged through a pressure regulator valve and the recycle distribution header. The finely divided air bubbles are released from the solution and begin the flotation action when the pressure is reduced. The flow through the flotation chamber is in a horizontal direction and the solids are skimmed continuously with flights and collected in a hopper. Figure 3 shows the influent end of the flotation chamber. Here the finely divided bubbles carry the solids to the water surface and the top-sludge blanket forms.

† Equipment manufactured by the Chain Belt Co., Milwaukee, Wis.

Pilot-Plant Operation

This pilot plant was installed and the operation was started on May 6, 1954. During the first two weeks operating personnel acquainted themselves with the operation of the unit.

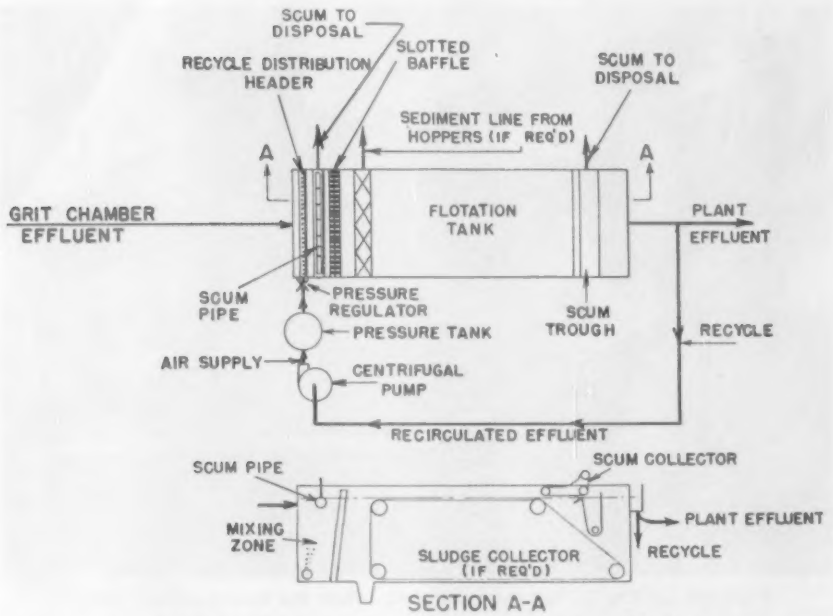

FIGURE 2.—Schematic diagram of the air flotation pilot plant.

A program of operation was then established and grab samples were collected. From these first determinations it was found that by observing the flotation action and scum density, a close estimate could be made of the efficiency of the unit's operation. Several difficulties as a result of the small size of the unit were also ironed out.

Following this break-in period the plant personnel knew what operation was wanted and the unit was put on a continuous run. Variations in operating procedure always were made at 7 AM, so that a 24-hr. test was possible with each setting of the air supply. Samples were composited on a 12-hr. basis, from 7 AM to 7 PM and from 7 PM to 7 AM. During the time of the test operation one of the wettest periods of weather was experienced since the Racine treatment plant was built. The night flows were so diluted that they were worthless in evaluating the operation of the unit. The suspended solids in the influent were often less than 50 p.p.m.

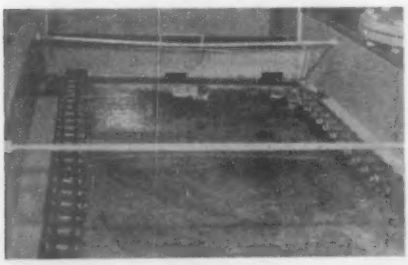

FIGURE 3.—Upstream view of the influent end of the air flotation unit.

Air Pressure

Several test runs were made varying the pressure under which the air was dissolved in the recirculated effluent. These variations in pressures ranged from 10 to 50 p.p.s.i. An operating range of 30 to 40 p.s.i. gave the best results, producing a satisfactory effluent and a sludge of good quality and quantity. When the pressure was reduced to 25 p.s.i. the quantity and density of the sludge was reduced slightly. A sharp reduction in performance efficiency was observed when the pressure was reduced to 20 p.s.i.

Recirculation

The recirculation rate and the detention time under pressure are interdependent. Because the volume of the pressure tank was fixed, reducing the recirculation rate increased the detention time in the pressure tank. The lower limit of the recirculation rate for satisfactory performance was 20 to 23 per cent of the influent flow. There was a sharp decrease in performance efficiency below 20 per cent. This percentage remained constant regardless of the flow rate. The upper limit was not as clearly defined, because at recirculation rates greater than 50 per cent the flow rate exceeded the design capacity of the pressure tank.

Detention

The graph (Figure 4) shows the effect of detention time and rate of air feed on the percentage saturation. These saturation values were obtained by volumetric measurements of the air supplied and the excess air leaving the pressure tank. The amount going into solution was calculated as the difference between the two measurements. Corrections were made for standard conditions.

To get good flotation, air saturation of 40 per cent or more is needed. This can be accomplished in two ways: (a) by using greater quantities of air or (b) by increasing the detention time. To reduce the detention time to one-half, would require two and one-third times as much air.

The effect of detention time under pressure as well as the amount of air injected, has a definite bearing on the degree of saturation. This is shown by the results obtained from injecting air at rates varying from 0.58 to 1.4 cu. ft. per 100 gal. of recirculated flow. With one minute detention time the amount of air in solution varied from 20 to 35 per cent of saturation. When the detention time was increased to 1.28 min., the amount of air dissolved was 37 to 63 per cent of saturation. With a detention time of 2.44 min., the amount of dissolved air was 36 to 97.5 per cent of saturation.

Test Data

This pilot plant was operated for eight months and considerable data were acquired. The unit was operated both above and below the designed capacity and when possible at a flow where solids separation was unsatisfactory. Because of the wet weather

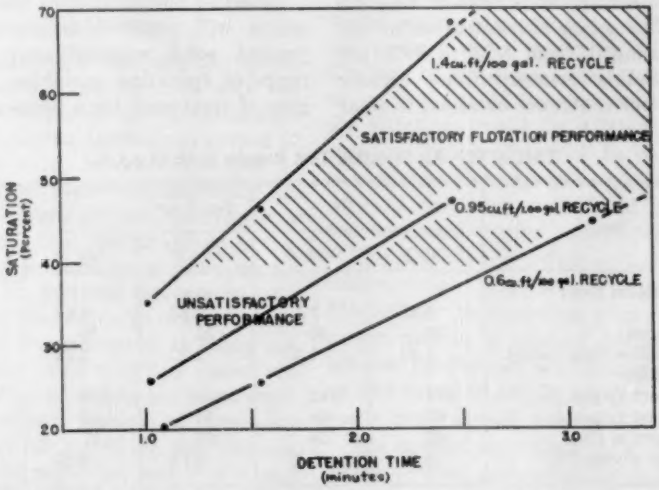

FIGURE 4.—Effect of detention time and air rate on percentage saturation of air in water.

TABLE I.—Air Flotation Test Results at 25 g.p.m.

Item	Test Runs[1]			
	June 18	June 7	June 8	June 11
Recirculated flow:				
Gal.	12.5	10.0	7.5	5.0
Per cent	50	40	30	20
Detention time (min.)	1.0	1.25	1.67	2.5
Susp. Solids:				
Influent (p.p.m.)	180	145	130	158
Effluent (p.p.m.)	63	70	59	93
Reduction (%)	65	52	55	41
Solids in sludge (%)	9.12	8.44	9.65	8.23

[1] Air feed is 10 cu. ft. per hour, air pressure is 40 p.s.i.

conditions and other reasons, only a part of the data will be given here for comparative purposes.

The data (Table I) were obtained from daytime samples with the unit operating at the designed capacity of 25 g.p.m. The detention time in the flotation zone varied from 15 to 19 min. depending upon the recirculation rate. The increased detention time of the recirculated effluent (1.0 to 1.67 min.) offset the effect of the reduced percentage of recirculation in the range of 30 to 50 per cent recirculation. At 20 per cent recirculation, a lower removal in suspended solids was observed, in spite of the compensating effect of the increased detention time in the pressure tank which was 2.5 min. The solids concentration of the sludge ranged from 8.23 to 9.65 per cent. Solids concentrations greater than 16 per cent were obtained in some tests. This concentration was too high to be handled easily under normal operating conditions.

The influent flow was reduced to 18.75 g.p.m. which increased the detention time in the flotation chamber. In the data shown in Table II, this detention time varied from 20 to 25 min. and the solids removal efficiency was good even at 20 per cent recirculation. This may have resulted from the longer detention periods of the recirculated flow in the pressure tank. The efficiency decreased with less than 20 per cent recirculation, but no values are shown because of difficulties in maintaining a constant recirculation flow at this rate for a 12-hr. period. Suspended solids removal varied from 53 to 64 per cent for the recirculation rates tested. The sludge concentration varied from 5.23 to 9.27 per cent solids.

The data given in Table III were obtained using an influent flow of 12.5 g.p.m. The solids removed varied from 56 to 74 per cent for the recirculation rates used. The percentage recirculation could not be held constant at less than 30 per cent because of the low flow. The sludge concentration ranged from 7.48 to 9.77 per cent solids.

Based on the conditions studied, flotation will produce satisfactory suspended solid removal over a wide range of operating variables. The degree of treatment for a particular sew-

TABLE II.—Air Flotation Test Results at 18.75 g.p.m.

Item	Test Runs[1]				
	July 1	June 23	June 25	July 15	July 14
Recirculated flow:					
Gal.	9.38	7.5	5.63	5.63	3.75
Per cent	50	40	30	30	20
Detention time (min.)	1.33	1.67	2.23	2.23	3.33
Susp. solids:					
Influent (p.p.m.)	171	142	148	103	128
Effluent (p.p.m.)	65	66	54	44	54
Reduction (%)	62	53	64	57	58
Solids in sludge (%)	8.48	9.27	6.72	5.23	5.77

[1] Air feed is 10 cu. ft. per hour, air pressure is 40 p.s.i.

TABLE III.—Air Flotation Test Results at 12.5 g.p.m.

Item	Test Runs[1]		
	June 28	June 29	June 30
Recirculated flow:			
Gal.	6.25	5.0	3.75
Per cent	50	40	30
Detention time (min.)	2.0	2.5	3.33
Susp. solids:			
Influent (p.p.m.)	185	177	183
Effluent (p.p.m.)	80	40	82
Reduction (%)	57	74	56
Solids in sludge (%)	7.48	9.77	8.69

[1] Air feed is 10 cu. ft. per hour, air pressure is 40 p.s.i.

age is dependent on the characteristics of the waste. It should be recalled that the influent to the flotation unit was obtained after grit removal; however, a small amount of bottom sludge was collected.

Tests for grit removal were conducted on both a laboratory and a pilot-scale unit by adding known amounts of 60-mesh grit. Almost 100 per cent recovery of the grit was obtained in the bottom sludge. The amount of putrescible material in this sludge was not measurable.

Conclusion

It was desired to compare the operation of this air flotation unit with the normal operation of the Racine sewage treatment plant. During the period of experimentation, however, the flow to the treatment plant was extremely erratic, as a result of flow interruptions caused by new construction work. These erratic conditions were aggravated by an unusual increase in precipitation. In order to make a comparison, it was necessary to study operating data from previous years. In 1953, the Racine treatment plant averaged 53 per cent removal of suspended solids with 68 p.p.m. in the effluent. In 1952, the reduction was 48 per cent with an effluent of 74 p.p.m. suspended solids. It may be noted that the removals obtained from the flotation unit were greater than these results.

During several of the test runs, analyses were made on the composite influent samples. The results of these tests showed an efficiency averaging approximately 10 per cent higher than the pilot plant. There are two probable reasons for this: (a) carry-over of sludge by the collector flights, or (b) insufficient attention by the operating personnel. The design of the skimming conveyor has been corrected. It should be mentioned that the flotation unit was operated by six of the plant personnel, who had regular duties to perform in addition to operating the pilot test unit.

The city of Racine is in the midst of a sewerage expansion program. One of the contemplated enlargements is a 50 per cent increase in clarification capacity. The demonstrated ability of the flotation unit to produce good suspended solids removals and highly concentrated sludge was considered with interest. The clarification facilities could possibly be expanded by building additional conventional sedimentation basins or by converting one of the four present clarifiers into an air flotation unit.

The cost of building two conventional sedimentation units similar to the present units would be $375,000. The cost of equipment and installation to convert one of the present units to air flotation would be $50,000. The present personnel would be sufficient to operate either equipment, so the additional operating cost would be only for increased power consumption.

The annual operating cost for two conventional clarifiers would be $10,500 for depreciation plus $34 for power making a total of $10,534. If interest of 2¼ per cent was included, an additional $8,437.50 would be added for a total annual cost of $18,971.50.

The annual operating costs for one of the present sedimentation units converted to air flotation would be $2,500 for depreciation plus $7,906 for electric

power, making a total of $10,406. If interest on the investment was included at 2¼ per cent, this would add $1,126, making the total annual cost $11,532.

The depreciation and power costs on the two methods of solids removal favor the conventional system by $94 per year. If interest on the investment is included, then the flotation system is favored by $7,439.50 per year.

The results of the pilot-plant operation previously described have demonstrated that removals of 50 to 60 per cent can be obtained over a wide range of operating conditions when used on municipal sewage. The higher operating costs are more than offset by the saving in construction costs.

Acknowledgments

The author wishes to express his gratitude for the assistance furnished by Stanley Budrys, Chemist, Racine Sewage Disposal Plant; A. C. Lind, Dr. W. J. Katz, A. Geinopolos and W. Throop.

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DISCUSSION

By A. W. Banister

Banister Engineering Co., St. Paul, Minn.

There is no doubt that the work at Racine (Wis.) relative to the flotation of municipal sewage presents an additional approach to many problems. Mr. Hay and his co-workers have gathered considerable data which should be very helpful to many designers.

However, several factors must be considered when the use of air flotation is considered. From the data presented it appears that quite a thorough knowledge or investigation of the wastes to be treated must be considered. Mr. Hay has commented on the variations of strength and the problems such variations present. Whether or not these variations are due to dilution by storm water is not clearly presented. Certainly the variations of flow from maximum to average and average to minimum might indicate this to be the case, as these variations are not as great as might be anticipated.

It appears quite conclusive that the per cent suspended solids reduction obtained at Racine is a function of the nature of the waste received by the flotation unit. Therefore, it may be anticipated that in the case of a waste containing suspended material not susceptible to air flotation the anticipated removals would be lower; conversely the presence of grease, oils, woolen mill waste, and other flotable wastes might provide higher removals.

Accordingly, the nature of the waste probably requires more consideration and study than would be the case for conventional sedimentation. It also appears that, with the high degree of quality operation provided at the Racine sewage treatment plant, the application of air flotation probably must be limited to the larger sewage treatment plants and that its application, generally will not be favorable in the smaller towns.

An additional consideration favorable to air flotation is the possible presence of residual dissolved oxygen in the effluent, thereby, reducing the quantity of chlorine required.*

The solids in the surface sludge re-

* Actually, the dissolved oxygen in the effluent from the air flotation unit varied from 48 to 90 per cent and averaged 59 per cent. Chlorine consumption was reduced 0.8 p.p.m. to maintain the same residual.

moved showed concentrations as high as 9.65 per cent and mention is made of concentrations up to 16 per cent. Perhaps such concentrations might be desirable in digestion, but the difficulty of handling this sludge will require special consideration in sludge pumping.

It appears that a minimum recirculation of 20 per cent and probably 25 per cent is required. At an air pressure of 30 to 40 p.s.i., under which the air is added for best results, pumping considerations are most important. Assuming 25 per cent recirculation and 30 p.s.i. pressure the recirculation pumps must provide 0.25 m.g.d. of waste against a head of about 82 ft. exclusive of any pipe friction loss.

It is to be noted that the economics given by Mr. Hay favor conventional units by $94 per year exclusive of interest. The paper also gives a favorable report of $7,439.50 per year for air flotation based on a 2¼ per cent interest rate. Such interest charges apparently are for the first year. If these interest charges were amortized over the life of the bond issue the saving offered by flotation would be reduced to $3,783.85 per year. This is still a substantial saving and warrants consideration.

The data presented deserve serious consideration. Perhaps careful evaluation of the results of an air flotation plant operating on a larger scale or full scale would provide additional data warranting a more favorable and wider usage of this method of primary clarification. The data so far obtained definitely present the need for further large-scale studies.

MAINTENANCE OF THE NEW YORK CITY INTERCEPTING SEWER SYSTEM *

By John J. Rooney

Civil Engineer, Sanitary, Department of Public Works, New York City, N. Y.

During 1953, the Intercepting Sewer Section was established in the Bureau of Sewage Disposal Operations to maintain and operate the interceptors and appurtenances that are under the jurisdiction of the New York City Department of Public Works. The addition of a number of new sewage treatment plants during the past few years put special emphasis on the importance of interceptor maintenance. It was deemed advisable to place a professional sanitary engineer in charge of this activity to standardize the operation and maintenance of the intercepting facilities. It also centralized the responsibility for having all dry weather flow carried to the treatment plants.

At the present time, this section maintains about 40 miles of intercepting sewers, 107 regulators and 185 tide gates diverting about 550 m.g.d. of sewage to 8 sewage treatment plants located in five boroughs.

Intercepting Sewers

The sewers in each borough of New York City are built and maintained by the various boroughs. Most of these are combined sewers that terminate between the bulkhead and pierhead lines in the waters surrounding New York City.

Interceptors are constructed by the Department of Public Works at lower elevations than the combined sewers and are expected to carry all sanitary waste during dry weather or a mixture of sanitary waste and storm water runoff equivalent to twice the mean dry weather flow. Usually when the

* Presented at the Metropolitan Section Meeting, New York Sewage and Industrial Wastes Assn.; New York, N. Y.; Nov. 30, 1954.

flow exceeds twice the mean dry weather flow, the excess is bypassed through the combined sewer outlets into the receiving waters.

Several inspections of the interceptors have been made because of blockages or fear of damage to them by adjacent new construction. Inspections are made during periods of low flow, usually at 2 or 3 AM, to minimize the quantity of sewage bypassed. After the sewage is bypassed by closing the regulators, a visual inspection is made of the interceptor, including the invert. Extensive deposits of stones, gravel, grit and sewage solids have been found but the interceptors appeared to be structurally sound. It is of interest to note that many of the stones in the interceptors weighed more than 25 lb. and had been carried several hundred feet from the nearest manholes by the flow. If the volume of flow increases to require the full carrying capacity of the interceptors, then the lines need to be cleaned.

Intercepting Sewer Components

Normally the Intercepting Sewer Section is engaged in the maintenance and inspection of the regulators and tide gates. To understand the operation of these devices, the function and interrelation of all the component parts must be known.

Combined Sewers

The combined sewers previously carried the sanitary and the storm flow from the drainage area to the receiving waters. The carrying capacities of many of these sewers have been decreased by extensive deposits of grit and sewage solids. When a new treatment plant is placed in service care must be taken to prevent excessive quantities of grit from reaching the treatment plant. Large quantities of grit may overload the new equipment and cause a shutdown almost before operation begins.

Diversion Dams

The diversion dams are placed across the combined sewers at elevations equal to the depth of mixed storm and sanitary flow at twice the mean ultimate dry weather flow. These dams divert the sewage through the regulator gates from the old discharge outlets to the stream. If the diversion dam elevation is below the high water mark of the adjacent receiving water, tide gates are provided, which vary in size from 12-in. diameter to 8 ft. by 10 ft. depending on the size and shape of the combined sewers. These gates prevent the backflow of tide water into the combined sewers, but they permit the bypassing of excessive storm water mixed with sewage which flows over the diversion dam and into the receiving waters.

Regulator Gates

A regulator mechanism is provided to limit the amount of mixed storm and sanitary flow entering the interceptor from each combined sewer. The mechanism consists of manually operated sluice gates or automatic sluice gates either hydraulically-activated or direct float-operated.

Manually-operated regulator gates are used where the combined sewer carries a very small quantity of sewage, thereby making automatic controls impracticable. Two gates are placed in series; each is supposed to be set so that the total head loss through both gates limits the storm and sanitary flow into the interceptor to twice the mean dry weather flow. The minimum opening actually used permits beer cans and similar trash to pass through so that these openings will not be obstructed too frequently.

Automatic sluice gates are controlled by a float located in a pit which is set at a level equal to the depth of twice the dry weather flow in the branch interceptor. During a storm the quantity of storm and sanitary flow in the combined sewer gradually increases as it passes through the regulator sluice gate until it exceeds twice the mean dry weather flow. Then the regulator float control rises and the sluice gate closes.

On the hydraulically-controled sluice gate, when the float rises, a 4-way valve is activated and permits water under pressure to enter the top of a cylinder forcing down a piston connected to the top of the sluice gate. The water from the bottom of the cylinder drains out through the 4-way valve. As the depth of the flow in the branch interceptor gradually decreases to less than twice the mean dry weather flow, the float moves down. This activates the 4-way valve, which opens the sluice gate by permitting water under pressure to enter the bottom of the cylinder, forcing the piston up. Water from the top of the cylinder flows back through the 4-way valve to waste.

The direct float operated sluice gate is moved up and down directly by the float, through a system of pulleys, shafts and cables or a walking beam.

This opening and closing, frequently referred to as hunting, continues until an equilibrium is reached where the storm and sanitary flow in the branch interceptor is equal to twice the mean dry weather flow. The float moves only slightly and the gate very little.

The excess storm water runoff which does not pass through the sluice gates backs up in the combined sewer until the water level exceeds the elevation of the diversion dam. Pressure is then exerted on the tide gates by the diverted flow. When the elevation in the combined sewer exceeds the tide level, the tide gate is forced open and bypassing begins. After the storm is over, the storm and sanitary flow in the combined sewer gradually decreases until the flow is less than twice the mean dry weather flow. The float falls, the regulator gate opens and the tide gate closes.

Branch Interceptors

A branch interceptor is the structure generally used to divert the sewage from the combined sewer to the interceptor. It connects the regulator chamber and the interceptor and is designed to permit the passage of twice the ultimate mean dry weather flow expected from a particular combined sewer.

Finally, the interceptor collects the flow from each branch interceptor and conveys it to the sewage treatment plant.

Routine Operation

As long as it does not rain sufficiently for the storm runoff plus the sanitary sewage to exceed twice the mean dry weather flow, there are very few maintenance problems with properly designed regulators. Therefore, the question to be considered is: how much runoff will result in twice mean dry weather flow? As an example assume that rain occurs at the Hunts Point Plant when the influent flow is equal to the mean daily dry weather flow. During 1953, the mean daily dry weather flow was about 97 m.g.d. or 150 c.f.s. The area tributary to this treatment plant is 15,613 acres. Therefore, a runoff of .0093 cu. ft. per second per acre would be equal to the mean dry weather flow. This is equivalent to one glass of water per acre per second. To determine the intensity of rainfall necessary to produce this runoff the equation used is $I = Q/CA$ in which $C = 0.5$ and $I = .02$ in. of rain per hour. Since the average time of concentration or collection time from this area is about 2 hr., .04 in. of rain in 2 hr. will produce a combined flow at the regulators equal to twice the mean dry weather flow. The regulators will commence operating and cause mixed sewage and storm water to be bypassed when the flow exceeds twice mean dry weather flow.

The United States Weather Bureau has recorded about five storms per month of more than this intensity during the past five years. It should

be noted, however, that rain occurring during periods of peak dry weather flow (time of peak sewage concentration) will produce only 50 per cent dilution of the sewage before being bypassed. On the other hand, rain at the time of minimum dry weather flow (time of minimum sewage concentration) would produce 150 per cent dilution before being bypassed.

During the rainfall and for some time thereafter, large quantities of debris are carried in the combined sewers because of the increased velocity associated with the increased flows. In addition, considerable extraneous material enters the sewers with the street washings and may interfere with the proper functioning of the regulators. Correcting any malfunctioning requires a visit to the regulator by one of the maintenance crews whose prime reason for this inspection is to stop possible sewage bypassing. An inspection may reveal an inoperative sluice gate caused by mechanical failure or by wood, rags or concrete debris obstructing the sluice gate or the branch interceptor. All of these conditions force the sewage to be discharged to the receiving stream. This discharge must be corrected if the flow of raw sewage to the harbor is to be minimized. The second reason for this inspection is to see that the tide gates have not been blocked open by wood or other debris caught between the flap and the seat. Such a condition would permit sea water to flow back over the diversion dam into the combined sewer and thence to the treatment plant, increasing the pumpage at the plant and decreasing the detention period in the treatment units.

The maintenance crews also repair the control equipment. Sluice gates and float controls must operate correctly to limit the storm flow to the treatment plant. Tide gates must be checked so that they will open and close properly. This requires extensive routine maintenance plus replacement or repairs to worn parts. Maintenance and repair work is done during periods of dry weather after all regulators have been checked and bypassing eliminated.

Special Operating Problems

There have been and still are many special operating and maintenance problems associated with the New York City sewer system. Solutions have been suggested by equipment manufacturers and the Bureau of Sewage Disposal Design. Many practical suggestions have come directly from the maintenance crews.

At several of the regulators, a considerable difference in invert elevation exists between the combined sewer and the intercepting sewer. The steep gradient exceeds the critical slope and results in high velocities and small depths of flow in the branch interceptor. Consequently, twice the mean dry weather flow is not sufficient to submerge the float.

Similar problems often occur when the regulator gate hunts and the flow level in the combined sewer rises above the tide elevation, exerting as much as 20 ft. of head on the sluice gate opening. As a result the discharge velocity increases. Although the depth of flow in the branch interceptor is controlled, the quantity of flow discharged is not controlled because of the high flow velocities.

In some cases, excessive velocities have been partially controlled by inserting horizontal stop planks downstream from the float pit to act as weirs. A 6-in. clear opening under the planks prevents the buildup of grit deposits behind them. The weir stop planks serve to convert some of the excess velocity head to hydrostatic head by increasing the depth of flow. However, if the approach velocity becomes too great because of exceptionally high bypass heads, the discharge over the stop planks will exceed twice

the mean dry weather flow. A trial installation has been made using vertical stop planks rather than the horizontal weir planks. The vertical plank control system has been more effective than the horizontal system and will eventually replace it.

Stainless steel is now being used for some of the moving parts in the 4-way valve. The original "Everdur" bronze parts were badly scoured by the water under pressure. The extra hard stainless steel appears to be more resistant than the bronze and will be used on future installations.

On the Rockaway interceptors, many of the small tide gates did not seat properly. This permitted the infiltration of large quantities of Jamaica Bay water. Tight closures were obtained by adding weights to the gates to force them shut and by using softer rubber gasket material, so that small surface irregularities would not effect the seal.

Although these tide gates were now properly sealed, the chloride concentration in the raw sewage at the Rockaway sewage treatment plant was still excessive, especially at high tide. Therefore, chloride determinations were made of the sewage at various locations on the separate sewer system. These field checks helped to locate numerous direct connections between storm sewers and the sanitary system which allowed bay water to flow from the storm sewer into the sanitary system at high tide. Most of these connections have been sealed.

Failure of the water supply pipe lines to the hydraulically-controlled regulators is also quite a troublesome maintenance problem. An example is the 1-in. brass line located in a subway yard that had to be replaced five times in 15 yr. This pipe failed due to electrolytic corrosion and various corrective measures were ineffective. The latest replacement was made with plastic pipe which is not affected by the electrolytic current and should solve the problem.

Special Studies

The Intercepting Sewer Section has also engaged in special studies in cooperation with the Laboratories of the Bureau of Sewage Disposal Operations. Some of these investigations were:

1. The determination of the relative concentration of wastes from the residential and industrial areas tributary to the Owls Head Sewage Treatment Plant.

2. A survey of the effect of combined sewage overflows on the coliform density in Jamaica Bay.

3. Industrial wastes surveys to locate the discharge points of industrial wastes which either interfere with the treatment processes or cause difficult working conditions at the sewage treatment plants. When complaints are received from one of the treatment plants, the maintenance crew examines the regulators to determine which combined sewer the waste may be flowing through. The treatment plant personnel are asked to log the day and time the objectionable material reaches the treatment plant. If a discharge pattern can be established, the maintenance crew stations itself at the regulator chamber prior to the time the objectionable waste is expected to reach this regulator. If a discharge pattern cannot be established, the maintenance crew is notified whenever the waste reaches the treatment plant. Then a check is made from manhole to manhole until the waste is traced to the offending industry. Since many combined sewers drain thousands of acres, these surveys often are tedious and time-consuming, yet essential to maintain proper functioning of the treatment processes.

Extensive flow studies also have been made to check design flow estimates for the Bureau of Sewage Disposal Design. Sewer record drawings are

checked first to obtain the cross-section and slope of the sewer at the section to be studied. The field survey crew determines the depth of flow at two adjacent manholes, the time of passage of either a dye or a float between the manholes, the dimensions of the sewer and the distance between manholes. Then the cross-section area, velocity and quantity of flow are computed.

If more accurate or additional information is needed, a pneumatically-controlled depth recorder is installed to record the depth of flow in the sewer. The volume of sewage is determined by applying the appropriate flow formula.

During one of these flow surveys, a simple method was found for diverting sanitary flow from the Coney Island Sewage Treatment Plant which is approaching design capacity to the Owls Head Sewage Treatment Plant which can readily accommodate this additional quantity of sewage.

Maintenance Crews

At the present time there are two maintenance crews composed of carefully selected men who service the intercepting sewer system. Each crew has a specially designed maintenance truck containing protected crew quarters, storage space for equipment and an engine-driven winch. Each truck is supplied with a variety of hand tools, a gas-driven generator, a gas-driven blower, life lines, a gas mask, explosimeter, portable lights, special clothing and tools, including an electric drill and hammer. One truck is stationed at the Manhattan Grit Chamber of the Wards Island Plant. The operating crew consists of one senior sewage treatment worker, 1 auto-engineman and 4 sewage treatment workers. The crew maintains and inspects 84 regulators and 101 tide gates on the Wards Island, Hunts Point and Port Richmond Systems. The second crew operates from the Owls Head Sewage Treatment Plant and has a crew of 4 sewage treatment workers and 1 auto-engineman. This crew maintains and inspects 23 regulators and 84 tide gates at the Owls Head, Rockaway, 26 Ward, Bowery Bay and Tallmans Island Sewage Treatment Plants.

Acknowledgments

The author appreciates the guidance and help of A. J. Scanlon, Bureau of Sewage Disposal Operations, Department of Public Works, New York City, and thanks Louis Fattore and the entire maintenance force in this section whose continued interest and cooperation have made these accomplishments possible.

TIPS AND QUIPS

Frank Woodbury Jones

The death of Frank Woodbury Jones, November 17, 1955 comes as a shock to those of the Federation membership to whom the Quarter Century Operators' Club and Frank Woodbury Jones were synonymous. It was he who created in 1941 this informal group of veterans of sewage treatment plant operation and, served as chairman and registrar of the club until 1954. In 1946 the club was given official recognition by the Federation Board of Control.

Though associated with the consulting engineering firm of George B. Gascoigne and Associates in 1923 and later (1940) becoming a partner in Havens and Emerson, Jones was perhaps best known as a sanitary chemist and long-time operator of sewage works. For several years he was chemist in charge of the Westerly Sewage Treatment Plant, Cleveland, Ohio. He was past-chairman (1952) of the Sanitary Engineering Division of the American Society of Civil Engineers. In 1952 he was the recipient of the Charles Alvin Emerson Medal for service to the Federation.

¿Habla Usted Español?

The 10th Edition of Standard Methods has been so well received that a third printing has been ordered. For those who have been unable to purchase a copy or have been reluctant to spend $7.50 ($6.50 to Member Association members) a Spanish volume is now available at $6.00. Copies may be obtained prepaid from Dr. Trois E. Johnson, Chief, HW & H Field Party, Institute of Inter-American Affairs, American Embassy, Mexico, D. F., Mexico. Comprenda?

Expensive Cuts

At the recent American Public Works Association Congress in Chicago, Ill., L. P. Staman, Director of Public Works, Tacoma, Wash., stated that U. S. street pavements are cut open for the installation or repair of some underground utility at a cost of $61,000,000 per year. All utilities are in need of more room both above and below ground, and he urged thinking ahead to coordinate and plan if there is to be any hope in eventually curbing the tremendous expense of undoing work already done.

Mr. Staman suggested that both public and private utilities and highway agencies participate in long-range planning discussions and that cities establish a single street-use ordinance to facilitate effective coordination. Subdivision control ordinances should be adopted which require that all underground installations be in place before acceptance of the plat. More uniformity in street opening regulations was recommended.

Quarter Century Operators' Club

The active membership roll of the Quarter Century Operators' Club was increased by the addition of two new members during 1955, although the passing of Julius W. Bugbee, Providence, R. I., brought a note of sadness. Seventeen members were present in Atlantic City for an excellent meeting.

Newly qualified members received at the Federation Luncheon were Nicholas De Haas, Sr., Linwood, Mass., and T. R. Haseltine, consulting engineer, Pittsburgh, Pa. The roster at that time included 47 living members and 10 deceased. In addition, several new applications were received at the meeting.

To be eligible for membership in the Quarter Century Operators' Club, the candidate must have been in full-time resident charge of the operation of a sewage treatment plant 25 years prior to date of his admission to the club. The increase in sewage treatment plant construction during the Federal construction program of the 1930's should result in a substantial increase in members during the next few years. Eligible persons are urged to contact the chairman of the club: Henry Van Der Vliet, P. O. Box 281, Rutherford, N. J.

Formal acceptance into the club is planned for the Annual Meeting of the Federation, but applications are invited at any time.

Federation Affairs

TWENTY-EIGHTH ANNUAL MEETING REPORT

The 28th Annual Meeting of the Federation, held at the Ambassador Hotel in Atlantic City, New Jersey, on October 10–13, 1955, attracted a record attendance of 1,345, and was among the top in program interest and success. Not only was the total registration the highest in Federation history, but the men's total of 1,051 and the ladies' total of 294 were both record highs. Geographically, members or guests were registered from 39 states, the District of Columbia, Alaska, Canada, England, Germany, Puerto Rico, and Switzerland.

The record breaking success of the meeting in all its phases is due to the effort and interest of the host New Jersey Sewage and Industrial Wastes Association, with the Local Arrangements Committee headed by Sol Seid of New Brunswick, N. J. The planning of details, the publicity and the cooperative efforts by the various local committees did much to assure the success of the meeting and the record registration. This will long be remembered as a milestone in Federation history.

The 28th Annual Meeting was convened at 10:30 AM on Monday, October 10, 1955 by President David B. Lee. Following the invocation, President Lee welcomed the attendants and commented briefly on this historic and active year of the Federation. The Executive Secretary-Editor's report, by Ralph E. Fuhrman, concluded the business portion of the opening session.

Technical Program

The technical program developed by the Program Committee, under the chairmanship of Professor Rolf Eliassen, presented topics of broad general interest, as well as many papers on specific pollution and waste treatment problems. Both the Industrial Wastes Forum and the Operator's Forum were received with their usual interest, although the lively discussions of the previous years were less evident.

A new forum, well received by technical personnel, featured current research activities at several universities. The Research Forum appears to have assured itself a place on future programs.

The technical program presented at Atlantic City was announced in the "28th Annual Meeting Preview" (see THIS JOURNAL, **27**, 9, 1102; Sept., 1955). Technical program papers are to be published in the 1956 issues of the Journal, beginning with this issue.

Highlight papers of the program included those presented by Dr. W. A. Patrick of Johns Hopkins University on mine drainage wastes and John J. Wirts on the proposal for a sludge transmission line to convey digested sludge from Cleveland and other cities in Ohio to the strip mine areas in the southern part of the state. A symposium on fringe-area sanitation was very timely and informative.

Entertainment Events

The featured entertainment of the meeting was new to Federation attendees. A carefree informal atmosphere prevailed during the Hallowe'en Country Night. Square dancing, games and contests made for a gala occasion.

The Federation Luncheon, held in the Embassy Room, was attended by approximately 400 Federation members and guests. W. H. Wisely, Ex-

ecutive Secretary, American Society of Civil Engineers, and former Executive Secretary-Editor of the Federation, brought personal greetings to those in attendance. He spoke on "The Federation as an International Force" and urged the membership to continue the international spirit at home and abroad. Mr. Wisely concluded with the showing of colored slides, taken on his tour of Europe in the summer of 1954 while on Federation business.

Several visiting engineers from Germany were introduced and members of the Quarter Century Operators' Club were recognized.

The Annual Federation Awards Dinner, held in the Renaissance Room on Wednesday evening, October 13, was attended by a record 878 members and guests. At the dinner, President Lee presented the annual Federation awards and introduced the new officers. Dancing followed to complete the enjoyable evening.

Ladies' Entertainment

The entertainment features planned for the ladies by the local Ladies' Entertainment Committee, under the sparkling chairmanship of Mrs. Ann Clark, were the subject of much favorable comment by the ladies attending the meeting.

Monday afternoon a relaxing and refreshing cocktail hour sponsored by the New Jersey Sewage and Industrial Wastes Association provided a warm and friendly atmosphere in which old acquaintances were renewed and new friends introduced. Attendance was marked by an increase in the number of younger women present.

On Tuesday an entertaining and educational feature enjoyed by all the ladies was the E. I. du Pont de Nemours Company's presentation of "Your Life and Man-Made Fibres." Many interesting and worthwhile tips were given on today's modern fabrics. The afternoon trip to the showroom of Lenox, Inc., well-known makers of fine china, was enjoyed by those who desired the opportunity of viewing various china patterns and making purchases.

Concluding event of the ladies' program was a delightful luncheon in the Marine Grill of Hackney's Famous Sea Food Restaurant on Wednesday. Ample opportunity was given for walking the famous boardwalk. The bright and sunny weather encouraged wading, sunning and beachcombing. Wednesday's activities were climaxed by the Federation Dinner and Dance.

Inspection Trips

Although no programmed inspection trips were included in the official program several interesting trips were made to industrial waste treatment facilities in nearby New Jersey. Trips to the organic and inorganic chemical works of Ciba States Limited and the frozen food processing facilities of Seabrook Farms drew good attendance.

Manufacturers' Exhibit

As a co-feature of the Atlantic City meeting the Water and Sewage Work Manufacturers' Association showed new equipment and process developments in an interesting and informative exhibit. All space alloted for displays was fully utilized to best advantage by the various manufacturers, this being the largest to date. from the standpoint of both the number of booths and exhibitors.

BOARD OF CONTROL MEETS AT ATLANTIC CITY

A historical and challenging year was concluded with the 28th Annual Meeting of the Federation Board of Control at Atlantic City. During the preceding 12 months the Federation office was successfully moved from Champaign, Illinois, to modern quarters in Washington, D. C. The new Executive Secretary-Editor, Ralph E. Fuhrman was adjusting to his many

new responsibilities. Some delays were noted in handling the routine Federation business and publication of the Journal as a result of the move and the need for employing new staff personnel. However, activities were smoothing out and satisfactory progress was noted.

Actions of the Board included the following:

1. Authorized the Sewage and Industrial Wastes Practice Committee to proceed with the publication of Manuals of Practice on Sewer Maintenance and Standardization of Units.

2. An entire revision of the Federation Constitution and Bylaws was recommended.

3. Revision of the Hatfield Award rules. In brief, Member Associations will be eligible to name recipients of these awards on a sliding scale based on the number of members enrolled on the previous September 30. Details of the arrangements are given elsewhere in this issue.

4. Approved the recommendations of the Research Committee that coordination of mutual research interests in the field of water pollution be furthered as much as possible through the Joint Committee for the Advancement of Sanitary Engineering, and that the U. S. Public Health Service be requested to develop as a part of its Research Program two or three master plans for high priority problems in the field of sewage and industrial wastes with Federation support.

5. Endorsement of the Water Pollution Control Bill before Congress, S. 890 as amended, was given by the Board of Control in line with the discussion during a conference of State and Interstate Water Pollution Control Administrators on this subject during the Atlantic City meeting. The Federation Board had previously approved the draft of the bill and the clarifying amendments have been proposed to answer certain objections expressed during the last year.

New Officers

The following officers were elected on October 9 for the terms indicated:

President: George W. Martin, Green Bay, Wis., to October, 1956.
Vice-President: John W. Cunningham, Portland, Oregon, to October, 1956.
Treasurer: W. W. De Berard, Chicago, Ill., to October, 1956.
Director-at-Large: Ray E. Lawrence, Kansas City, Mo., to October, 1958.

Experience outlines for the new President and Vice-President are given elsewhere in this issue.

1955 Awards

The following honors and awards, as conferred by the Board, were presented or announced by President Lee at the Annual Awards Dinner on October 12:

Elected to Honorary Membership was Gordon Maskew Fair of Cambridge, Massachusetts, renowned educator of sanitary engineers at Harvard University and internationally known in the field.

The Harrison Prescott Eddy Medal was earned by W. L. Lea, G. A. Rohlich and W. J. Katz of the University of Wisconsin for their research contribution "Removal of Phosphates from Treated Sewage" (THIS JOURNAL, **26**, 3, 261; Mar., 1954).

The George Bradley Gascoigne Medal was conferred on A. H. Chasick for his plant operation contribution entitled "Activated Aeration at the Wards Island Sewage Treatment Plant" (THIS JOURNAL, **26**, 9, 1059; Sept., 1954).

The Industrial Wastes Medal was awarded to Kenneth S. Watson of the General Electric Company, Schenectady, N. Y., for his contribution "Treatment of Complex Metal-Finishing Wastes" (THIS JOURNAL, **26**, 2, 182; Feb., 1954).

For his numerous scientific contributions to the technical literature and

in particular for the development of the present standard B.O.D. determination and for his outstanding record as a teacher and inspiration to the Central States Sewage and Industrial Wastes Association, M. Starr Nichols was awarded the 1955 Charles Alvin Emerson Medal.

Nominations for the 1955 Arthur Sidney Bedell Awards, for service to Member Associations of the Federation, were confirmed by the Board as follows: C. H. French (Arkansas); Fred G. Nelson (California); George F. Schroepfer (Central States); Carl E. Schwob (Federal); Leo Holtkamp (Iowa); F. T. Hambleton (ISP, England); Herman D. Regan (Kentucky-Tennessee); James L. Love (Louisiana); Thomas J. Powers (Michigan); William F. Rapp, Jr. (Nebraska); Henry Van Der Vliet (New Jersey); R. C. Sweeney (New York); P. D. Davis (North Carolina); Emil Jensen (Pacific Northwest); T. R. Haseltine (Pennsylvania); W. T. Linton (South Carolina); Nils Westberg (Sweden); and J. H. Sorrels (Texas).

Membership

The Secretary reported that the aggregate Active and Corporate membership of the 38 associations comprising the Federation was at a record high of 6,693 on September 30, 1955, a gain of 7.4 per cent for the year. The numerical increase of 491 during the year was the highest in the history of the Federation for a year without a gain in number of Member Associations.

The Michigan association, with an increase of 100 members, won the numerical increase contest over the valiant efforts of the Florida (78) and Georgia (76) associations.

The Georgia association took the percentage increase prize with a 145 per cent increase following a close race with the Nebraska association (108 per cent). Other contenders for this prize included the Florida (43 per cent) and Michigan (32 per cent) associations.

Financial

The operating budget for fiscal 1956 is balanced at $112,250, an increase of almost $10,000 over the previous year. Provision is made for all necessary foreseeable expenses in the Federation's new quarters.

Audited net worth of the Federation at the end of 1954 was $75,802.14. Once again a modest increase is indicated for the year 1955.

Detroit in 1958

Meeting places for the years 1958 through 1960 were approved by Board action. Previously announced meeting sites already approved were Los Angeles in 1956 and Boston in 1957. Officially designated as future meeting places were:

1958, Detroit, Michigan, Sheraton-Cadillac Hotel, Oct. 5–9.
1959, Dallas, Texas, Hotel Statler Hilton, Oct. 11–15.
1960, Philadelphia, Pennsylvania, Philadelphia Sheraton, Oct. 2–6.

New invitations received verbally from the Florida Sewage and Industrial Wastes Association, the Georgia Water and Sewage Association (Atlanta), the Missouri Water and Sewerage Conference (Kansas City) and the Verband Schweizerischer Abwasserfachleute (Swiss) were referred to the Meeting Place Committee. Invitations from Member Associations were already on hand from: Canadian Institute (Toronto), Central States (Milwaukee), Rocky Mountain (Denver) and Pacific Northwest (Portland or Seattle).

HATFIELD AWARD RULES CHANGED

As a result of the considerations of the Plant Operations Committees, the Board of Control empowered the Executive Committee to act finally on the Committee's recommendation. (On December 19, 1955, the Executive Committee voted to follow the recommendations of the Plant Operations Committee.) Accordingly, the William D. Hatfield Award will be revived and the frequency of this Award for meritorious operating reports will be made according to the size of the Member Association as follows:

1. That the number of awards per Member Association be as follows:

0 to 34 members	By petition only
35 to 249 members	1 per 3 years
250 to 499 members	2 per 3 years
500 to 749 members	3 per 3 years

but that if any Member Association in the 0 to 34 class increases its membership by 50 per cent and is still under 35 members, it shall by petition be favorably considered for an award before reaching the 35-member status to allow eligibility for one nomination in a 3-year period.

2. That no Member Association nominate more than one recipient per year.

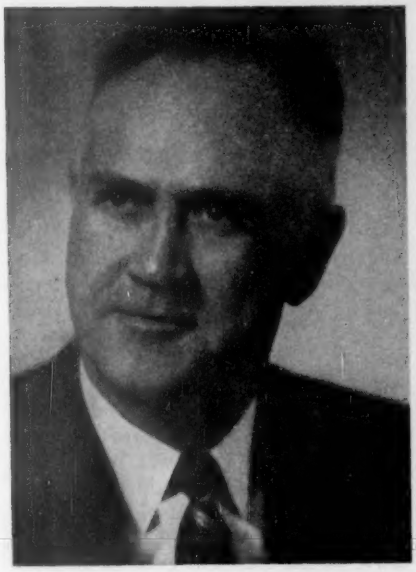
3. That the awards for the succeeding year be made in accordance with the accompanying schedule and reviewed as to eligibility as of the preceding September 30.

Hatfield Award Schedule

Member Association	Members 9-30-55	Awards per 3 Years	Future Schedule	
			1956	1957
Alabama	35	*	—	†
Arizona	38	1	—	1
Arkansas	72	1	—	—
California	575	3	1	1*
Canadian Institute	442	2	1	1*
Central States	653	3	1	1†
Dakota	30	*	†	—
Federal	80	1	—	—
Florida	260	1	—	1
Georgia	128	1	1	—
Germany	39	1	—	1
Iowa	113	1	—	—
ISE (England)	27	*	—	†
ISP (England)	129	1	—	—
Kansas	116	1	—	1
Kentucky-Tennessee	120	1	—	—
Louisiana	8	*	—	—
Maryland-Delaware	72	1	1	—
Michigan	415	2	—	*
Missouri	46	1	—	1
Montana	33	1	1	—
Nebraska	79	1	—	—
New England	369	2	1	1*
New Jersey	226	1	—	—
New York	617	3	1	1*
North Carolina	142	1	—	—
Ohio	356	2	1	1*
Oklahoma	47	1	1	—
Pacific Northwest	290	2	—	1*
Pennsylvania	506	3	1	1*
Puerto Rico	37	1	1	—
Rocky Mountain	69	1	1	—
South Carolina	44	1	—	—
Sweden	80	1	—	—
Swiss	15	*	—	†
Texas	198	1	—	—
Virginia	115	1	1	—
West Virginia	72	1	1	—
Total	6,693	—	15	14

* Conditional upon continuing membership.
† Year eligible to petition for one nomination.

GEORGE W. MARTIN NEW PRESIDENT



George W. Martin, Chief Engineer and Superintendent, Green Bay (Wis.) Metropolitan Sewerage District, became the 16th President of the Federation at the organization meeting of the 1956 Board of Control, held at Atlantic City, New Jersey, on October 13, 1955.

Following graduation from the University of Wisconsin in 1926 with the Bachelor of Science degree in Engineering, Mr. Martin was employed as resident engineer with a Wisconsin engineering company. During the period 1926 to 1935 he was engaged in a number of municipal construction projects on paving, sewers and sewage treatment. In 1935 he accepted his present position of Chief Engineer and Superintendent of the Green Bay Metropolitan Sewerage District, a newly completed construction project on which he had been resident engineer.

Mr. Martin is a Registered Professional Engineer in Wisconsin and has been active in professional engineering societies. He was president of the Wisconsin Society of Professional Engineers (1948–9) and a Director both to the Wisconsin Society of Professional Engineers (1941–2) and the National Professional Engineers (1950–2).

He has been interested in the welfare of sewage treatment plant operators, having served the Conference of Wisconsin Sewage Works Operators (President, 1941). In addition, he has served Central States Sewage and Industrial Wastes Association as President (1941–2) and Federation Director (1948–51).

Mr. Martin is also a past president of the Green Bay Engineers Club and served on the Green Bay Water Commission (1938–52). He was the recipient of the Federation's Kenneth Allen Award (1946) and the William D. Hatfield Award (1949). In 1933 he received the degree of Civil Engineer from the University of Wisconsin.

Besides serving the Federation on various committees and as the Central States Director, Mr. Martin also served as Vice-President of the Federation in 1954–5. He has been a member of the Subcommittee on Sewage Treatment Plant Design (1953–4), and the Executive and Meeting Place Committees of the Federation. At present he is a member of the Executive, Finance, Joint Policy, Meeting Place and Honorary Membership Committees.

JOHN W. CUNNINGHAM ELECTED VICE-PRESIDENT

John W. Cunningham, Consulting Engineer, Portland, Oregon, was elected to the Vice-Presidency of the Federation at the Election Committee meeting on October 9, 1955.

As a consulting engineer of many years experience on water and sewage works, he comes to his Federation office with a particularly good insight into and an earnest appreciation of the objectives and aims of the Federation.

Prior to the 1935 reorganization of the consulting engineering firm that now bears his name, Mr. Cunningham had been engaged in structural steel design, irrigation and reclamation work throughout the Pacific Northwest. Since then he has been active principally in the design of works for the collection, purification and distribution of water as well as the collection, treatment and disposal of sewage. He holds a Bachelor of Science in Engineering degree from the University of Wisconsin (1908) and is a registered Professional Engineer in several states.

Mr. Cunningham has long been active in the affairs of professional engineers. As a member of the Oregon State Board of Engineering Examiners from 1935 to 1953 he served as President (1950–3) and as a representative on the National Council of State Boards of Engineering Examiners. Besides holding membership in several general engineering societies he is a member of the Pacific Northwest Sewage and Industrial Wastes Association, the Professional Engineers of Oregon, the American Water Works Association and Tau Beta Pi. He has served the Federation as Director of the Pacific Northwest Association (1940–2). Currently, he is a member of the Executive and Meeting Place committees of the Federation.

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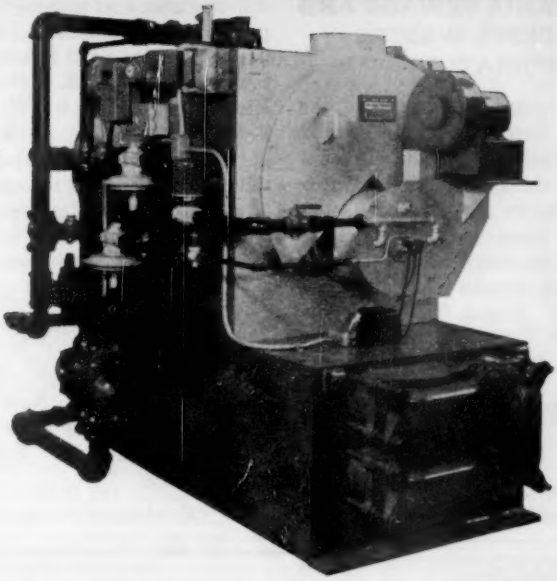

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Proceedings of Member Associations

PENNSYLVANIA SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

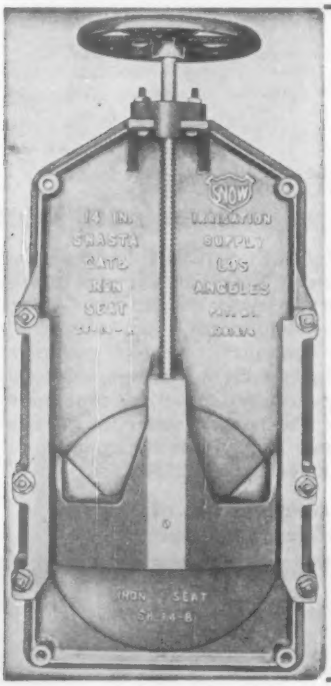
The 29th Annual Conference of the Pennsylvania Sewage and Industrial Wastes Association was held, in conjunction with the Pennsylvania Water Works Operators' Association, at Pennsylvania State University, State College, Pa., on August 22–24, 1955.

The program opened with the Sewage Operators' Round Table, moderated by Frank Varner, Altoona. Participants were John F. Bowder, Lititz; Robert Perry, Meadville; James Volk, Penn Township; Clarence Cooper, Conshohocken; and J. N. Hagerich, Grove City. This was followed on Tuesday by a Sewage Works Symposium on handling sludge from various treatment processes. Papers were presented by Albert L. Genter, Baltimore; Robert F. Stutzman, Tullytown; Elwood R. Harding, Sr., Bloomsburg; Ralph W. Woodring, Bethlehem. Robert M. Bolenius, Abington and Dewnaye D. Day, Meadville, discussed the laboratory control of an activated sludge treatment plant.

Papers of particular interest to industrial wastes personnel included:

"Control of Sludge Volumes Following Lime Neutralization of Acid Wastes," by Samuel D. Faust, Research Assistant, Dept. of Sanitation, Rutgers University, New Brunswick, N. J. Discussion by G. M. Neas, Staff Engineer, American Viscose Corp., Philadelphia.

(*Continued on page 31a*)



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"Methods for Determining the Treatability of Industrial Wastes," by Professor Charles E. Renn, Dept. of Sanitary Engineering and Water Resources, Johns Hopkins University, Baltimore, Md. Discussion by Roy F. Weston, Sanitary Engineer, Newtown Square, Pa.

"Treatment of Medicinal and Fine Chemical Wastes by High Rate Activated Sludge," by F. E. Dryden, Waste Treatment Dept. Head; P. A. Barrett, Associate Research Chemical Engineer; J. C. Kissinger, Supervisor, Waste Treatment Dept.; and J. Lago, Sr., Research Chemical Engineer, all of Merck & Co., Danville, Pa. and W. W. Eckenfelder, Jr., Weston, Eckenfelder and Hood, Paterson, N. J. Discussion by J. A. Liontas, Waste Control Engineer, Sharp & Dohme, Philadelphia, Pa.

"The Collection and Treatment of Industrial Wastes at the Rohm & Haas Company, Bristol, Pennsylvania," by J. H. Steen, F. M. Majewski, and Thomas Iezzi, respectively Sanitary Engineer, Development Engineer, and Assistant Sanitary Engineer, Rohm & Haas Co.

"Conventional Digestion and Digestion with Gas Recirculation," by James H. Blodgett, Manager, Sewage Treatment Plant, Dept. of Public Service, Columbus, Ohio. Discussions by T. H. Forrest, Chicago Pump Co., Chicago, Ill.; George Crawley, Superintendent, Sewage Treatment, Erie, Pa.; and J. R. Harvey, Pennsylvania Dept. of Health, Meadville, Pa.

Officers were elected as follows:

President: Lewis L. Gwin. Altoona.
1st Vice-President: George A. Elias, Philadelphia.
2nd Vice-President: Bernard S. Bush, Wilkes-Barre.
Secretary-Treasurer: J. R. Harvey, Meadville.

(*Continued on page 32a*)

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FSIWA Director: Roy F. Weston, Newtown Square.

BERNARD S. BUSH,
Secretary-Treasurer

MISSOURI WATER AND SEWERAGE CONFERENCE

The 1955 Annual Meeting of the Missouri Water and Sewerage Conference was held at the Connor Hotel, Joplin, Mo., on September 25–27, 1955 in conjunction with the Missouri Section, AWWA. Registration of members and guests totaled 230.

The program featured discussions on water pollution control and its effect on water and sewerage utilities. Meeting participants resolved to sponsor water pollution control legislation and authorized committees of the two

(*Continued on page 33a*)

SIX (6) TOP SUPERVISORY POSITIONS AVAILABLE IN THE CITY OF MIAMI'S NEW SEWAGE TREATMENT PLANT

The City of Miami, Florida, is accepting applications to staff their new Activated Sludge Sewage Treatment Plant, to be completed during the early part of 1956, as follows:

SEWAGE TREATMENT PLANT SUPERINTENDENT. (1) Salary range $7,872–$9,744. Desired Qualifications: 8–10 years previous experience in the operation of complex sewage treatment works. Education: Degree in Sanitary Engineering; or in Civil or Chemical Engineering with courses in sanitary engineering.

ASSISTANT SEWAGE TREATMENT PLANT SUPERINTENDENT. (1) Salary range $6,372–$7,872. Desired Qualifications: 6–8 years previous experience in the operation of complex sewage treatment plants. Education: Degree in Sanitary Engineering or in Civil or Chemical Engineering with courses in sanitary engineering.

SEWAGE TREATMENT PLANT SUPERVISORS. (4) Salary range $5,388–$6,648. Desired Qualifications: At least five (5) years previous experience in the operation of complex sewage treatment plants. Education: Graduation from high school or a vocational school with courses in the chemistry and biology of sewage treatment or related subjects.

The above positions are all under Civil Service jurisdiction and include paid vacations and sick leave, health, welfare and pension plans and many other employee benefits. Excellent working conditions in a wonderful climate with congenial employees.

Send complete summary of your education, previous experience and personal data to C. F. Wertz, Director, City of Miami, Department of Water and Sewers, 3342 Pan American Drive, Miami 33, Florida. (All replies treated strictly confidential.)

associations to prepare a suitable bill and enlist public support.

Papers presented at the joint sessions of water and sewage works personnel were as follows:

"Soil Mechanics—Useful Tool in Water and Sewage Plant Development," by Henry M. Reitz, Professor of Civil Engineering, Washington University, St. Louis, Mo.

"How to Build Good Public Relations," by Robert A. Willier, Public Relations Consultant, St. Louis, Mo.

"Stream Pollution and Its Control on the National Level," By Gordon E. McCallum, Chief, Water Supply and Water Pollution Control Program, USPHS, Washington, D. C.

"Stream Pollution and Its Control in the Missouri River Basin," by Glen Hopkins, Assistant Regional Engineer, Water Supply and Water Pollution Control, USPHS, Kansas City, Mo.

"Proposed Stream Pollution Legislation in Missouri," by Robert C. Smith, State Representative, Boone County, Mo.

"Stream Pollution Control in Missouri," by Albert W. Happy, Jr., Director, Bureau of Public Health Engineering, Missouri Division of Health, Jefferson City, Mo.

In addition, papers on biofiltration, gas recirculation, mechanical aeration and the use of concrete were presented at the Sewage Division Sessions.

Charles A. Haskins was presented with the Federation's Arthur Sidney Bedell Award at the annual banquet by Ralph E. Fuhrman, Federation Executive Secretary.

Officers elected to serve during 1955–6 were:

Chairman: Edward G. Lee, Lexington.
Vice-Chairman: Frank McAndrew, St. Joseph.
Secretary-Treasurer: Warren A. Kramer, Jefferson City.

W. A. KRAMER,
Secretary-Treasurer

DIRECTORY OF ENGINEERS

(Continued through page 39a)

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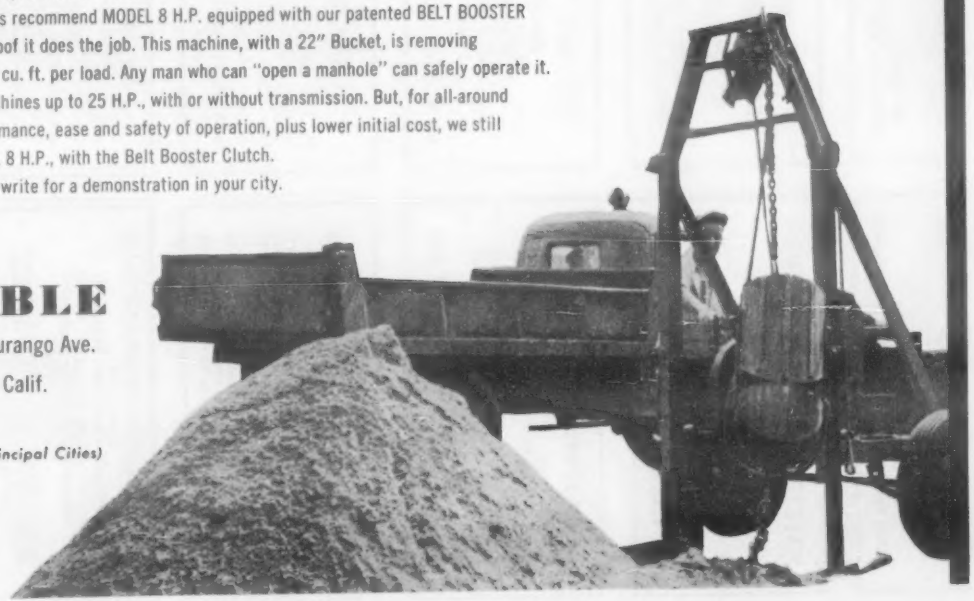

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INDEX TO ADVERTISERS

	Page
American Well Works	18a
Armco Drainage and Metal Products, Incorporated	23a
Bailey Meter Company	22a
Builders-Providence (B-I-F Industries, Incorporated)	3a
Cast Iron Pipe Research Association	8a, 9a
Chapman Valve Manufacturing Company	14a
Chicago Pump Company	1a
Clow, James B. & Sons	17a
Combustion Engineering, Incorporated (Raymond Division)	16a
Dorr-Oliver, Incorporated	28a
Everson Manufacturing Corporation	31a
Flexible Incorporated	40a
Infilco Incorporated	7a
Jeffrey Manufacturing Company	25a
Kennedy-Sanderson Engineering Corporation	20a
Link-Belt Company	15a
Lock Joint Pipe Company	21a
Morse Brothers Machinery Company	24a
National Clay Pipe Manufacturers, Incorporated	13a
Pacific Flush Tank Company	26a
Portland Cement Association	19a
Rockwell Manufacturing Company (Nordstrom Valve Division)	12a
Simplex Valve and Meter Company	5a
Snow Gates and Valves	30a
Tacmec Company, Incorporated	33a
United States Pipe and Foundry Company	11a
Vapor Recovery Systems Company	10a
Walker Process Equipment Incorporated	29a
Wallace & Tiernan Incorporated	Back Cover
Zimmer and Francescon	32a

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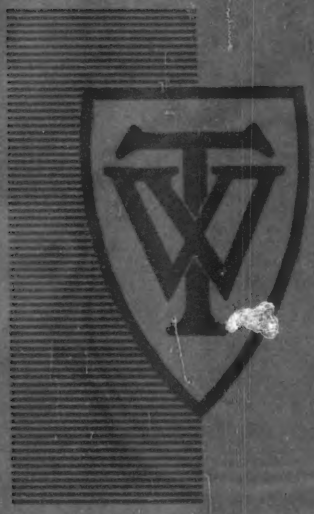

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